RATIONAL LOAD FACTORS AND DESIGN OF PRECAST PRESTRESSED PORTAL FRAME

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BY
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CERTIFICATE

This is to certify that the thesis entitled 'RATIONAL LOAD FACTORS AND DESIGN OF PRECAST PRESTRESSED PORTAL FRAME' by T. Sanjeeva Reddy is record of work carried out under my supervision and has not been submitted elsewhere for a degree.

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LIST OF SYMBOLS

A_{st}	=	Area of tension reinforcement
A_{se}	=	Area of compression reinforcement
C_c	=	Cost of concrete, compression force in concrete
C_s	=	Cost of steel
C_u	=	Total internal compression at ultimate load
C_m	=	Compression force in mild steel
D	=	Total depth of section
F_b	=	Beam load factor
F_j	=	Joint load factor
F_c	=	Column load factor
F_o	=	Overall load factor
M	=	Bending moment or moment of resistance
$(M_{wx})^u$	=	Working load moment at x of upper beam
$(M_{wx})^l$	=	Working load moment at x of lower beam
$(M_{pb})^u$	=	Plastic moment capacity of beam (upper)
$(M_{pj})^l$	=	Plastic moment capacity of joint (lower)

M_u	=	Ultimate moment
P	=	Axial load
P'	=	Equivalent eccentric load
P_d	=	Dead load
P_p	=	Peak load
P_w	=	Working load
P_u	=	Ultimate load
P_u^E	=	Experimental ultimate load
P_u^T	=	Theoretical ultimate load
T_m	=	Tensile force in mild steel bars
T_p	=	Tensile force in prestressed wires
T_u	=	Total internal tensile force at ultimate load
a	=	Depth of rectangular stress block
b	=	Width of rectangular section
d	=	Effective depth of section
d_c	=	Concrete cover
d_o	=	Total depth of section
e_u	=	Eccentricity at ultimate moment
f'_{cy}	=	Cylinder strength of concrete at 28 days

n = Depth of rectangular stress block

p = Steel ratio

q_c = Allowable shear stress

ϵ_{se} = Tensile strain in reinforcement due to effective prestress

ϵ_{su} = Tensile strain in reinforcement at ultimate moment

ϵ_{scu} = Compressive strain in reinforcement at ultimate moment

ϵ_u = Ultimate compressive strain in concrete

γ = Design coefficient of loads

σ_{cu}, f'_c = Cube strength of concrete at 28 days

σ_{se} = Effective prestress in reinforcement

σ_{su} = Tensile stress in reinforcement at ultimate moment

σ_{scu} = Compressive stress in reinforcement at ultimate moment

σ_{sy}, f'_{sy} = Yield stress of steel in tension

σ'_s, f'_{su} = Ultimate tensile strength of steel

θ = Virtual rotation of the hinge

ϕ = Capacity reduction

<u>NOTE</u>: Superscript p and m refer to prestressing and mild steels respectively.

SYNOPSIS

A brief review of the literature on precast concrete connections and the effect of repeated loads on reinforced and prestressed concrete structures has been presented. The need for introducing load factor design, a concept of different rational load factors to different elements depending on the extent of uncertainties in their design assumptions and difficulties in construction has been discussed.

A theoretical study has been made on the effect of variable joint load factor relative to the beam load factor on the strength and cost of the structure with reference to a single bay multistoreyed frame. It is found that the strength increases upto a certain level of joint load factor and remains constant while the cost continues to increase with the joint load factor.

p

Experimental investigation has been carried out with a view to study the behaviour with reference to strength and serviceability of precast prestressed portal frames with variable joint load factors under the action of pulsating loads varying from fixed load level to working load level with occasional peak loads.

Based on the theoretical study and experimental investigation different load factors, 1.4 to 1.5 for beams, 1.7 for columns and 1.9 to 2.1 for joints have been recommended. Load factors for the elements have to be chosen to obtain an overall factor of 1.8 to 2.0. At higher load levels greater than 75% of ultimate loads, the structure has been found to lose its serviceability in view of developing wide cracks though its strength not affected. Stiffness of the frames has been found to reduce with the increasing number of pulsations, the reduction being lesser for the higher values of joint load factors.

INTRODUCTION

1.1 Purpose of a Structural Design :

The purpose of a structural design is to provide a structure complying with user's requirements, paying appropriate attention to overall economy, safety, servi- ceability and aesthetics. The 'economic factor' implies that the investment covering both the first cost and subsequent maintenance cost should be minimum. The provision for 'safety' requires that the risk of failure of the whole or part of the structure should be suffi- ciently small during its specified life. 'Serviceability' implies that the structure has to retain an appearance not disquieting to the user and general public, that the cracks if any be kept to tolerable limits and that vibrations be minimized. The provision of aesthetics stipulates that the completed structure should fit into the environment and generally be pleasing to the eye.

Review of present design procedures:

In general there are three basic approaches to the design of concrete structures,

i) working stress design ,

ii) ultimate strength design and ,

iii) limit state design .

1.2 Working Stress Design :

This is a design criterion based on the permissible stresses, which are obtained on the basis of idealized elastic behaviour of steel and concrete. The permissible stress in concrete is a certain fraction of the cube strength or cylinder strength and in steel it is a fraction of yield stress or ultimate stress. The ratio which the yield stress of the steel or cube strength of concrete bears to the corresponding permissible or working stress is called the 'stress factor of safety' or more briefly the 'Factor of safety'. The members are so proportioned, that under working load conditions on the structure, the maximum stresses in concrete and steel do not exceed the permissible stresses. The working load is taken as the sum of actual dead loads of the structure and an estimated maximum live load. The live loads also include the normal wind loads. But when the loads such as storm loads earth-quake forces and their combinations act on the structure, it will be overstressed beyond the permissible stress values.

Most of the codes allow higher permissible stresses for the combined action of the loads as the probability of such a combined load occurrence is rather small.

Permissible stresses adopted by the various codes of practice are given in Table 1.1. Columns 3 to 6 in the table give the allowable stresses under normal loads and column 7 gives the higher permissible stresses to be adopted for the combined loads or overloads. These loads, which occur rarely, may be due to the combined action of wind, earthquake and other forces or due to the accedental overloads in the normal loading itself. This provision of extra allowable stresses in some codes is arbitrary and is not based on the knowledge of the extent of actual overloading or combined loading, its frequency and its effect on the structure. It is therefore desirable to have a statistical basis for the assessment of the extent of overloading or combined loading beyond the normal load level and the frequency of its occurence on various types of structure, for a more rational design.

TABLE 1.1

ALLOWABLE STRESSES ADOPTED BY VARIOUS CODES OF PRACTICE

Sl. No.	Code of Practice	Allowable Stress in Concrete: Max. Fibre Stress in Compn.	Allowable Stress in Concrete: Max. Fibre Stress in Tension	Allowable Stress in Concrete: Max. Stress in direct Compn.	Allowable Stress in steel in tension	Higher permissible Stress in case of combined load
1	2	3	4	5	6	7
1.	I.S.I(1)*	0.33 to 0.35 f'_c	0.033 to 0.035 f'_c	0.24 to 0.27 f'_c	0.5 f'_{sy}	33-1/3% more
2.	ACI (2)	0.45 f'_{cy}	$1.6\sqrt{f'_{cy}}$	0.25 f'_{cy}	0.5 f'_{sy}	33-1/3% more
3.	Russia (6)	0.50 to 0.55 f'_c	0.035 to 0.055 f'_c	0.40 to 0.44 f'_c	-	-
4.	B.S.I.(4)	$\frac{1}{2.73} f'_c$	-	$\frac{3}{4} \times \frac{1}{2.73} f'_c$	0.55 f'_{sy}	25% more
5.	CEB.FIP** (7)	$\frac{1}{1.5} f'_c$	-	$\frac{1}{1.5} f'_c$	$\frac{1}{1.15} f'_{su}$	-

Note : ** Factors given by CEB-FIP must be considered in combination with load factors.

* Figures inside the bracket indicate the Reference number given in the Appendix.

f'_c : cube strength of concrete at 28 days

f'_{cy} : cylinder strength of concrete at 28 days (f'_{cy} = 0.75 to 0.80 f'_c)

f'_{sy}: yield stress of steel in tension

f'_{su}: ultimate stress of steel in tension.

The working stress design gives no information regarding the actual factor of safety against the failure of a reinforced concrete member. However, it does give a realistic representation of conditions in a member at working load and therefore is useful in predicting the behaviour characteristics such as deflections, crack widths etc. There are other drawbacks in the working stress design. The basic assumption in the design process that the materials behave elastically is not true especially at the higher ranges of stress values. Further the presumption that a single factor i.e. the so called 'stress factor of safety' takes care of all the possible factors responsible for the destruction of a structure is not rational.

1.3 Ultimate Strength Design :

This method is based on the more complete utilization of the actual properties of both steel and concrete. The members are designed based on the conditions just before the failure of the structure. They are so proportioned that the full strength of the cross section is just utilised when the ultimate load is applied. The ultimate load on the structure is obtained by multiplying the working load by a factor known as the 'load factor'.

Since this method permits the accurate evaluation of the strength capacities of the members, it predicts the safety factor against the failure. But this gives little information about the behaviour of the structure at the working load conditions.

The most important deficiency of the ultimate strength and as well as the working stress design methods is that in these methods no direct account has been taken of the variability of material strength in the completed structure and also the variability of anticipated loads on it. The material strength in the completed structure may be less than that found from samples due to various reasons such as the inherent variability of the material arising from its method of manufacture, corrosion of steel during the life of structure, sustained and repeated loading etc. The loads may be greater than those anticipated, due to the possible increase in the general level of loading in the structures of the type considered, errors of construction increasing the dead weight, and effects of shrinkage, creep or temperature changes etc. Considerable emphasis must therefore be placed on a statistical treatment of both the strength of materials and the loading. A more rational approach incorporating the above factors is made in the limit state design.

1.4 Limit State Design :

This method differs from the preceding two in having strictly defined limiting states for members and by introducing several design factors or coefficients instead of a general factor of safety. A structure is said to be fit for use if its strength and safety are within certain limits. The limit state is therefore defined as the state at which the structure or part of it ceases to fulfil the function for which it was designed. In any structure unfitness for use may arise in various ways. The most important of them all are as follows :

Collapse : This is defined as occuring when the loads acting on a structure exceed its load carrying capacity. The collapse may arise from rupture of one or more critical sections, loss of overall stability, or buckling due to elastic or plastic instability.

Excessive Deflection (or displacement) : This is defined as occuring when the deflection impairs the appearance or efficiency of structure, affects adversely the non-load bearing members or causes discomfort or alarm to the users of the structure.

Excessive Local Damage : This occurs when cracking or spalling of concrete impairs the efficiency or appearance of the structure.

When any structure is rendered unfit for use for its designed function by one or more of the above causes, it is said to have reached a limit state.

1.5 CEB Approach :

Comite European Du Beton (CEB) in their 'recommendations for an international code of practice for reinforced concrete' (7) have suggested some sort of semiprobability approach. Since all the data necessary for a rigorous probability approach are not available, the CEB has suggested the introduction of 'characteristic values' of the strengths of materials and loads on the structure. These values are based on a fixed probability that the actual values will be either less or greater than the values selected and will be obtained from the statistical normal distribution. To cover the remaining uncertain factors, these characteristic values are transformed into design values by introduction of certain coefficients.

The characteristic strength σ_K of a material is given by one or the other of the two relations: $\sigma_K = (\sigma_m - KS)$ or $\sigma_K = \sigma_m(1-K\cdot\delta)$ where

σ_m is the arithmetical mean of the various experimental data ,

S is the standard deviation ,

δ is the relative mean quadratic deviation and

K is a coefficient depending on the accepted probability of the test results being below the value of σ_k.

The design strength $\sigma^* = \sigma_K / r_m$ where r_m is the strength reduction coefficient which is the function of the statistical laws concerning the errors or faults which occur during construction.

The characteristic value of the loading Q_k is given by $Q_k = Q_m\ (1+K\delta)$ where Q_m is the value of the most unfavourable loading, with a 50% probability of its being exceeded, upto abnormally high values, once in the expected life of the structure; δ is the relative mean quadratic deviation of the distribution of the maximum loading; K is a coefficient depending on the accepted probability of Loadings greater than Q_k.

The design load Q* is given by $Q^* = r_s Q_k$ where r_s is a coefficient depending on the probability of a particular limit state being reached, possible increases in the permanent or superimposed loads, possible errors in construction, probability of several loads not occuring at the same time and the possible redistribution of stresses.

1.6 Limit Design of Statically Indeterminate Concrete Structures :

It is an inelastic theory of statically indeterminate concrete structures in which readjustments in the relative magnitudes of internal moments and forces at various sections are recognised at high loads. The limit of load carrying capacity of the structure is obtained when the mechanism is developed after the sufficient number of plastic hinges are formed on the basis of full redistribution of moments. This is somewhat analogous to the plastic design of steel structures, the main difference being in the two important aspects mentioned below.

Rotation Capacity : The ultimate strain capability of concrete is as low as 0.3 to 0.7% compared to about 15 to 20% in a steel section. Therefore a reinforced concrete

section has limited rotation capacity and it varies to some extent with the ratio of reinforcement in the section. Under-reinforced concrete members controlled by yielding of the tensile steel, permit considerably higher rotation compared to over-reinforced concrete sections. This limited rotation capacity results in excessive cracking unless adequate safeguards are provided. If such cracking occurs at loads very near to ultimate loads, it may be harmless with regard to the serviceability. But the excessive cracking at or slightly above the design loads is objectionable.

Distribution of Moment Resistance : By varying the amount and location of reinforcement, the positive and negative moment of resistance of structural concrete members can easily be made different, and it can also be varied conveniently along the length of a prismatic member. It is therefore convenient to reinforce the concrete structure, in a manner that the distribution of moments at ultimate load level is reasonably close to the moment distribution corresponding to elastic behaviour. All possible hinges necessary to perform a mechanism will then form practically at the same load and therefore the hinge rotations are very small.

1.7 Precast Construction :

There is a marked tendency in the recent years towards the increasing use of precast construction, especially in building industry. Precast construction is faster than conventional methods, provided the earth-works, foundation work and financing keep pace with this fast construction method. It is also cheaper than monolithic construction provided the production of elements is mechanised involving the manufacture of them in mass scale.

Prestressing in precast construction is also being increasingly adopted to reducethe cost and weight of structure. The general practice is to prestress flexural and tension members and to use the ordinary reinforcement in the compn.members. Sometimes precast compression elements are also prestressed to take care of the secondary stresses likely to develop while handling the members.

Joints in Precast Construction :

The joints in precast construction are provided mainly for continuity of the structural system. These joints may be made by various methods such as welding the

steel reinforcements or structural steel inserts, bolting, use of key type devices; use of bonding mediums, use of friction between the members induced by gravity forces, prestressing or by combination of any of the preceding methods. Design of individual members of precast construction is reasonably simple, but the design of efficient and economic joints is one of the major problems. Since these joints directly affect the integrity of the structure in which the precast members are used, their design as well their execution must be adequate for the function intended.

An ideal joint should be capable of being designed to transfer all the imposed loads with a known margin of safety. It should withstand the loads without marked displacement or rotation and avoid high local stresses. It should accommodate tolerances in the elements, require little temporary support, and demand only a few distinct operations to make in its fabrication. It should be proof against the deterioration caused by the exposure to weather and admit effective inspection and rectification. It should be neat and compact avoiding awkward shapes and intrusion into the living space. The ACI (11) has recommended that the ultimate strength of joints and connections in precast construction should be atleast 10 percent in excess

of that required of the members connected.

1.8 Response of a Structure to a Lateral Loading :

The usual types of lateral loads are wind loading, earthquake loading, crane drag force, blast loading etc. The effect of all these lateral loads should be invariably considered in the strength and stability analysis of the structure. In addition certain serviceable conditions play an important role in the design of a structure for lateral loads. The most significant service criteria are as follows:

Lateral Drift : This is a relative magnitude of lateral displacement at top of building with reference to its height and has a significant part in the design of high rise buildings. ACI (46) recommends a deflection limit of 1/500 for wind loads.

Relative Vertical Deflection : The effect of relative vertical deflection between the exterior and interior columns and between the columns and shear walls needs careful examination.

Cracking : Cracking of non-structural elements such as partitions, windows etc., may cause serious maintenance

problems and so the drift limitation should be selected to minimize such cracking.

Lateral Sway motion : The sway motion of a tall building under turbulent wind, if perceptible, may produce psychological effects which render the building unfit from user's point of view. The design of structure for the lateral loads should provide for the reduction of such perceptible motion to acceptable levels.

1.9 Simulated Loads on a Structure :

The working load on a structure includes the dead load and an estimated value of live load. Wind loads, seismic loads and all the gravity loads other than dead load constitute the live load. But the inherent nature of variability of all these loads makes it a difficult task to assess a fairly accurate value of the live load. Seismic load occurs rarely depending on whether the structure is located in a seismic zone or not. The wind forces are highly fluctuating and their variation is extremely random. A typical case of the variation in wind velocities is shown in Fig. 1.1. The variation of other live loads on a structure also will be random, especially in case of bridges where the traffic will be changing from

time to time varying from no traffic load conditions to the peak traffic. A typical random variation of the overall loads on the structure including the dead load, liveload, wind load, earthquake load and any other possible combination of the same is shown in Fig. 1.2.

Since it is not possible to design a structure to suit this random variation, an idealization has to be made. A typical idealised load frequency diagram is shown in Fig. 1.3. The object of this idealization is to select a suitable design load for the structure such that in its life time, it will not fail due to the occasional heavier over loads on combined loads coming on it, nor it will end in an overdesign affecting the economy. The loading level B in the diagram appears to be the reasonable choice of a design load. The peak load frequencies at the level C in the diagram are called overloads or combined loads as these may be the result of the combined effect of wind and seismic loads. In addition to the choice of a design load, a fairly reasonable estimate of the magnitude and frequency of these combined loads is also essential, to assess the serviceability of the structure at these peak loads.

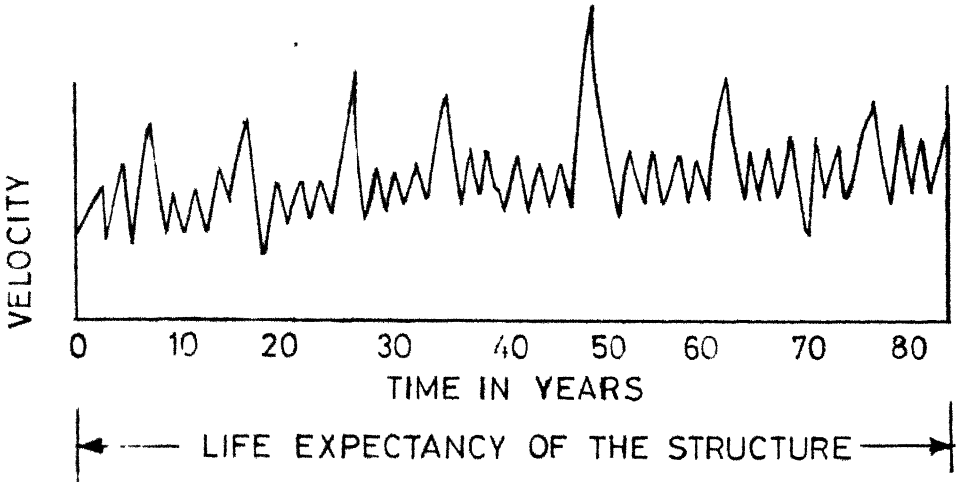

FIG.1.1 TYPICAL VELOCITY CHARACTERISTIC OF WIND

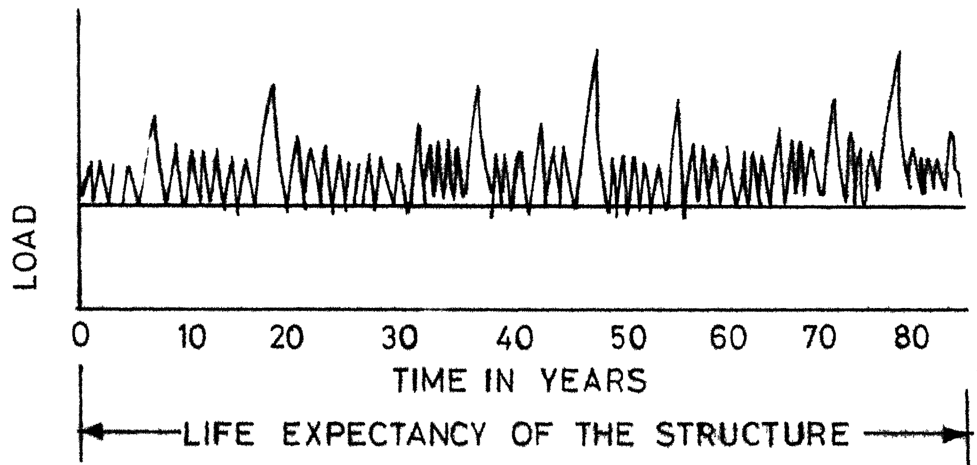

FIG.1.2 TYPICAL LOAD CHARACTERISTIC

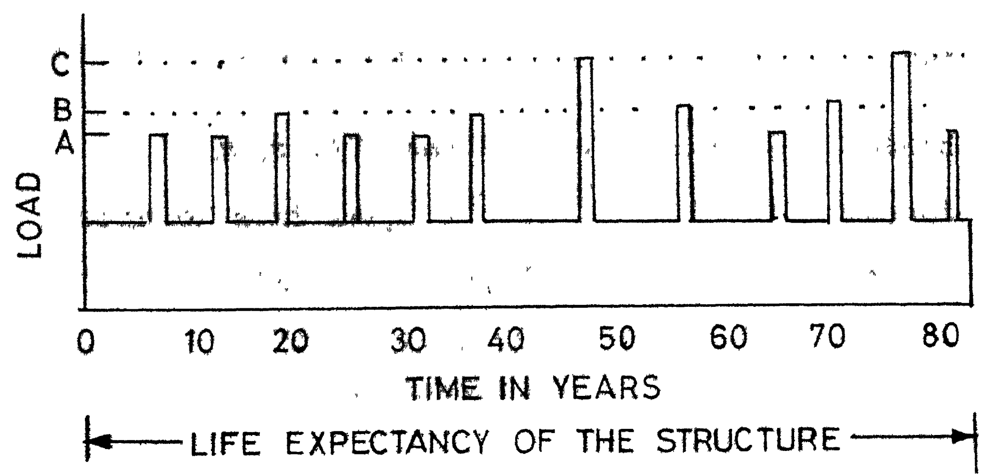

FIG.1.3 IDEALISED LOAD CHARACTERISTIC

1.10 Review of Literature :

As the object of the present investigation is the study of the behaviour of prestressed precast portal frame under simulated loads a brief review of the literature on the precast concrete connections and on the repeated loading effects on reinforced and the prestressed concrete structures is presented.

Joints in Precast Concrete Connections :

ACI-ASCE Committee 512 in its report (11), has recommended various methods by which the joints for use in precast concrete construction may be designed and constructed. PCI Committee for connection details (12) have brought out a booklet giving the various types of connections between precast prestressed elements. PCI have also developed (13,14 & 15) certain empirical design criteria based on the experimental results. Gerfen (16) has presented several methods in which precast wall panels may be tied together to make a precast building.

Mast (17) has described a simple method for the design of auxiliary reinforcement in the precast as well as in the cast-in-situ concrete connection, based

on the study of a physical model. Using the shear friction hypothesis Birkeland (18) has developed a design criterion for the beam-column joints. Hanson and Connor (19) have conducted experimental investigation to determine the joint reinforcement required to give the adequate strength under the repeated reversed loads simulating the earthquake forces. PCI seismic committee (20) have outlined certain general principles for the design of connections to resist the earthquake forces.

Effect of repeated loading on the behaviour of prestressed and reinforced concrete structures :

Repeated loads and mode of failure :

When all the loads acting on a structure symultaneously increase proportionally, the structure is said to be acted upon by 'proportioned loading'. When they increase and decrease proportionally several times it is the 'repeated loading'. When the repetition of load is in quick succession it is termed as pulsating load. In actual practice different loads on a structure may not satisfy the condition of proportionality and they may change in magnitude independently from each other. These non-proportional loads, when they vary several times are called

'variable repeated loads'. Sometimes these loads may change in their direction also resulting in the occurence of the reversal of stresses at the critical sections of the structure. Such loading condition is termed as 'Repeated reversible loading'. Although, all the above types of loads act in a random manner, a cyclic pattern of the occurence of these loads is generally assumed when the effect of these loads is considered.

Different modes of failure that may result from repeated loading are 'fatigue', 'alternating plasticity' and 'incremental collapse'. Fatigue is a failure of the material by fracture as a result of repeated loading on the structure. When the failure is by yielding of the material alternately in compression and tension at a given cross section, the phenomenon is called 'alternating plasticity'. This term is more precisely used for steel structures. The structures may fail by an increase in deflection during each cycle of loading, the increments of deflection being in the same direction. This method of failure is termed as 'incremental collapse'. In the repeated application of loads the increments in deflection may cease after a few cycles of load application. When the structure reaches this stage of stabilized deflection it is said to have reached a 'shake down limit'.

Mattock (22), Earnst (23) and others have shown that if properly designed, reinforced concrete structures can have sufficient rotation capacity to allow full redistribution of stresses assumed in the inelastic theories of the design of reinforced concrete structures. But their assessment was based only on proportional monotonically increasing load tests. Further investigation has therefore been carried out by various authors to study the effect of variable repeated loading including the reversible repeated loading on the rotation capacity and other aspects of the reinforced concrete structures.

Behaviour of Concrete under repeated loading :

To arrive at a rational basis for predicting the response of a structure to varying loading, it is necessary to obtain the information about the mechanical behaviour of material under this type of loading. Sinha, Gerstle and Tulin (24) obtained stress-strain curves for cyclic loading based on the tests on cylinders, and arrived at certain analytical stress-strain relations. Assuming the property of the uniqueness of the stress-strain relation, they have also shown how the cyclic stress-strain curves can be used to predict the behaviour of

concrete fibre subjected to an arbitrary load history. Shaw B.P. (25) reports that cyclic or sustained stresses on concrete result in the progressive internal micro crack propagation. Murashev et al (6) stated that the ultimate strength of concrete decreases with the stress ratio ($\sigma min/\sigma max$), under the action of repeated loads.

Repeated Loads on R.C. Beams :

Response of reinforced concrete beams to the repeated loading has been investigated by a few authors. Gerstle et al (26,27) have investigated the response of singly and doubly reinforced concrete beams to variable repeated and as well as reversed loading. A bending theory of R.C. beams to arbitrary cyclic load histories has been developed based on the stress-strain relations of concrete under cyclic loading mentioned earlier, and compared with the test results. They have concluded that the response to repeated loading may be considered elastic plastic for engineering purposes but the behaviour under reversible load is highly non-linear.

Verna and Stelson (28,29) have conducted tests to determine whether there are any significant changes in

the beam strength and modes of failure as the load history changes and concluded that

i) beam specimens designed for steel yielding failure under single cycle static load gain the static ultimate strength for a history of about 100,000 cycles, (ii) beams designed for diagonal tension failure gain the strength upto about 18000 cycles and tend to lose for greater no. of cycles, iii) specimens designed for bond failure are most susceptible for damage under repeated load and they become weaker with increasing number of repeated load cycles and (iv) the mode of failure under repeated load depend on the load level and designed mode of failure under static load. Since the bond consideration plays a significant role in the behaviour of reinforced concrete beams under repeated load, Ruiz (30) has used deformed bars with a short yield plateau and found that neither rotation nor load carrying capacity of a reinforced concrete beam is affected by several lakhs of cycles of near ultimate loading and that deformations are moderate increasing at a decreasing rate, becoming negligible after a few cycles.

Structures subjected to earthquake movement undergo reversible loadings and the possible progressive damage that may occur in R.C. beams under such loading is a subject of vital interest. N.H. Burns (31) has investigated this effect and found that due to repeated reversible load strength of beams was not reduced but there was a loss of ductility. The elastic stiffness decreased almost linearly upto about one half of its initial value and remains practically unchanged. Strain hardening effect of steel cannot be ignored in any realistic prediction of the flexural response either to static load or repeated loading. Gerstle et al (32,33) have investigated this effect on the under-reinforced concrete beams and suggested that it may be possible to disregard the effects of possible cycles of overload upon the flexural strength of R.C. beams, in view of the beneficial effects of strain hardening.

Prestressed Concrete Beams Under Repeated Load :

A few investigations (34,35,36) are reported on the effect of repeated load in prestressed concrete beams and the findings are listed below:

i) At about 50% of the static single cycle ultimate load level of repeated loading the majority of the beams could be expected to survive as many as 5 million cycles. Those failing in fatigue are likely to fail by strand fatigue although compression fatigue is also possible.

ii) At 60 and 70 percent load levels only few specimen survived 5 million cycles, and the majority of fatigue failures are due to strand fatigue.

iii) At 80 and 90% load levels specimens survived only a few hundred cycles and the fatigue failure will be mostly in compression.

iv) Flexural stiffness will decrease and permanent set increases with increased repetition of loads.

v) Flexural tensile cracks will migrate towards the top of the beam with increased repetition of the load. At low load levels crack progression terminates after several million load cycles.

Sawko (35) suggested restressing of the tendons as a remedy to improve the flexural stiffness. Price (36) has presented a theory for the prediction of fatigue

strength of prestressed concrete beams which is applicable for repeated loading. Coles and Hamilton (37) have investigated the effect of low cycle highly impulsive repeated load on pretensioned beams and concluded that the calculated ultimate static load is a satisfactory design criterion for pretensioned beams subjected to low number of blast type of loadings. Ozell and Diniz (38) have investigated the effect of repeated loading on composite beams (cast-in-situ over precast prestressed) and found that provision of shear ties between the two surfaces is essential for an effective composite action under repeated loads.

Patnaik (39) has investigated the effect of pulsating loads in a prestressed concrete beam at the working load level with occasional overloads and concluded that a prestressed concrete beam with a load factor of 2 and with overloads as high as two thirds the ultimate load capacity will have satisfactory serviceability. Ultimate strength is not affected much even after a million cycles of pulsation. The beams may survive a limited number of overloads as high as three fourths of the ultimate load.

Reinforced Concrete Frames under Repeated and Reversible Loads :

Very few investigations (40,41,42,44) are reported on the effect of repeated and reversed loading on the R C Frames. Bertero and McClure (40), Beaufait (41), and Sabnis (42), have conducted tests on the frames to investigate the behaviour both under repeated cyclic gravity loads and repeated reversed lateral loads and have arrived at the following conclusions under repeated gravity loads :

i) Low cycle repeated loading as high as 95% of single cycle static ultimate load capacity, has no significant effet on the load carrying capacity of the frame.

ii) Reduction in the stiffness of the frame is not considerable.

Under repeated reversible loads :

i) Ultimate load capacity is essentially unaffected upto about 75% of non-alternating single cycle ultimate load, stiffness is slightly reduced and bond strength at critical sections is reduced to some extent.

ii) At about 80% load level as low as 10 numbers of reversed load cycles produced extensive damage. Frame strength was reduced considerably, cracking was severe, heavy damage to the bond strength at critical section and a sharp reduction in the stiffness was observed.

Gowda (43) has investigated the effect of pulsating loads on prestressed precast portal frames under the vertical loads at working load level with occasional overloads of about 33-1/3% in excess of working load and found that the strength and serviceability with reference to cumulative deflection is not affected.

1.11 Object of Study :

The present practice of adopting same load factors for all the elements of a structure appears to be less reasonable. Selection of rational load factors to different components of a structure depending on the extent of uncertainties in their behaviour and design, needs a careful study. This concept assumes more importance in case of precast construction, in view of the inevitable differences in quality control in the construction of precast and cast-in-situ portions of the structure.

Especially joints are vulnerable in this method of construction. It is therefore proposed to make a theoretical study of the effect of variable joint load factor relative to the beam load factor on the overall strength and economy of the structure. A single bay multistorey frame is selected for this study. It is also proposed to investigate experimentally the effect of variable joint load factor relative to beam load factor on the behaviour of precast prestressed portal frame with the object of arriving at rational load factors for the beams and joints in a precast construction.

Hitherto the emphasis of investigations was on the behaviour of structures to monotonically increasing loads in general. But in actual practice the loads on the structure will be varying at random from the lowest load levels of the dead load to the highest load levels pertaining to the combination of live loads, wind loads earthquake loads etc., probably several million times in the life span of the structure. Very few investigations have been carried out in these fields and much less in precast concrete construction. Therefore it is proposed to investigate experimentally, the behaviour of a prestressed precast portal frame under the action of vertical

and horizontal loads representing the gravity loads and normal wind loads respectively, when subjected to repetitive cyclic loadings varying from dead load level to working load level several lakhs of times, reaching occasionally the peak load level representing the combined load level and study the behaviour with reference to the following:

i) Change in the load carrying capacity of the structure due to repetitive cyclic loading.

ii) load versus deflection behaviour before and after the cyclic loading.

iii) Change in the stiffness of the structure after the cyclic loading.

iv) study of the pattern of crack formation and

v) behaviour of the structure under different combined load levels near the ultimate load.

CHAPTER II

LOAD FACTORS

2.1 Introduction :

The ratio of design ultimate load to the working load of an element is called 'ultimate load factor' or more briefly 'load factor'. Most of the codes specify the values for load factors. But it is the designer's responsibility to choose a load factor appropriate to the conditions for which the particular structure is designed, with due consideration to the probability of failure and consequences of failure. The necessity for the provision of load factor in the design arises out of the sources of errors in the design data assumed and other safety requirements. Different load factors have to be chosen for different types of loads such as dead load, live load, impact load, snow load, strain loads such as due to temperature variation and shrinkage, lateral loads such as wind or earthquake and for different combinations of the above loads. They may also have to be varied with the nature of load whether dynamic or

fatigue producing and with the type of structure whether a bridge, building, tower or a tank. The load factors are used both in the ultimate strength design as well as in the limit state design. In the ultimate strength design they represent the ratio of ultimate load to the working load, the ultimate load being associated with the critical section of each member rather than that of the structure.[*] In the limit state design the load factor has been adopted as a design coefficient to account for the possibility of the service load being exceeded because of its variability. Load factors adopted by various codes of practice for concrete structures and for plastic design of steel structures are given in Tables 2.1 and 2.2 respectively.

2.2 Provision of different load factors to different elements in a structure :

In the present design practices the load factors are based on the type of load and are independent of the type of the structural element. For example all the components of a framed structure viz., beams, columns, slabs and joints are given the same margin of safety by using the same load factor. One of the important considerations in the selection of load factor is the possible

LOAD FACTORS ADOPTED BY VARIOUS CODES OF PRACTICE FOR CONCRETE STRUCTURES

Sl.No.	Code of Practice	Design Ultimate Load	Remarks
1.	I.S. CODE OF PRACTICE (1)*	1.5 D + 2.2L or 1.5D + 2.2L + 0.5(Wor E) or 1.5 D + 0.5L + 2.2 (Wor E) Whichever gives critical conditions	
2.	American Concrete Institute (3)	1.4D + 1.7L or 0.75 (1.4D + 1.7L + 1.7 W) or 0.9 D + 1.3 W	These factors are considered in combination with capacity reduction factors of the section (0.90 for flexure, 0.75 for compn.etc.)
3.	British Standard Code of Practice (4)	1.4D + 1.6L or 1.25D + 1.25L + 1.2 (Wor E) or 1.4 D + 1.4 W	
4.	German Standards (5)	1.75 D + 1.75 L	
5.	Russian Standards (6)	(1.1 to 1.2) D+ (1.2 to 1.4)L	These factors are for the ultimate limit state and shall be considered in combinations with strength reduction factors applied to the ult. stresses.
6.	CEB - FIP International Recommendation (7)	1.5 (D+L) or 0.9D+1.5L	- do -

D: Dead Load, W: Wind Load, L: Live Load, E: Earthquake force.

* Figures in bracket indicate the reference number given in the appendix.

TABLE 2.2

LOAD FACTORS FOR PLASTIC DESIGN OF STEEL STRUCTURES IN VARIOUS COUNTRIES (8)

Sl.No.	Country	Assumed shape factor	For Dead load plus live load	For Dead load + live load + wind load + Earthquake force
1	U.S.A.	1.12	1.70	1.30
2	Australia	1.15	1.75	1.40
3	Belgium	1.12	1.68	1.49
4	Canada	1.12	1.70	1.30
5	Germany	-	1.71	1.50
6	India	1.15	1.85	1.40
7	Mexico	1.12	1.70	1.30
8	South Africa	1.15	1.75	1.40
9.	Sweden	-	1.57	1.34
10.	U.K.	1.15	1.75	1.40

inaccuracies in the basic design assumptions and design methods. But the extent of assumptions and uncertainties in the design is not same for all the elements in a structure. For example the design of column involves more uncertainties compared to those in the beams, owing to the buckling effect and the difficulty in assessing the amount of end restraints. It appears therefore less reasonable to adopt the same margin of safety to beams and columns.

The structural elements in a precast construction are produced in the factories with all the necessary care and supervision to obtain their designed strengths. But when those elements are transported to the site and assembled, it is very difficult to exercise the same quality control as in the factories, while casting the joints within the reasonable economical constraints. Casting of joints will be difficult due to the invonvenient positions at which they have to be cast. Also the congestion of reinforcement at the joint due to the presence of reinforcement from beam and column makes it difficult to cast a good joint. Therefore the reliability of the joint

is less compared to the elements. The fact that the joint may have lesser reliability is also true to some extent even in the monolithic construction, due to lack of thorough understanding of the behaviour of the joint.

Due to the above factor, it appears more rational to adopt different load factors to different components of the structure depending on the extent of uncertainties associated with the elements. This concept of different load factors for different elements, which is known as 'load factor design' is under active consideration for steel structures. ASCE in its 'commentary on plastic design of steel structures (8)' envisages a factor of 1.67 for tension members, short columns and slender beams, 1.92 for long columns and 1.70 for compact shapes. In case of reinforced concrete structures provision of different load factors for different components of the structure can be done very easily by altering the reinforcement with or without changing the cross sectional dimensions of the members.

2.3 Effect of variable joint load factor on the overall safety of the structure :

The joint load factor plays an important role

in the load factor design of reinforced concrete structures. Under the pattern of loading most generally adopted, joints constitute some of the critical sections in a framed structure and some of the plastic hinges will be formed near the joints at collapse mechanism conditions. By suitably increasing the joint load factor, thus increasing the moment capacities of the section near the joint, the overall load factor of the frame will be increased. The overall load factor is the ratio of the design ultimate load to the working load on the structure, the ultimate load being obtained with different load factors for the elements. The increase in the overall load factor represents the increase in the strength of the structure and with predetermined fixed working load this reflects an increase in the safety level of the structure. The increase in overall load factor F_o with the increase in joint load factor F_j depends upon the order of indeterminacy of the structure and the lengths of portions upto which the increased moment capacity at the joints extends into the beams and columns.

To study the effect of variable joint load factor, single bay single storeyed, two storeyed, three storeyed

and fourstoreyed portal frames are considered. Keeping the beam load factor constant, joint load factor is varied and the overall load factor is obtained. As a typical example the detailed analysis of a single bay two storeyed frame is given below.

2.4 Analysis of a single bay two storeyed frame with variable joint load factor :

The dimensions and the loading particulars of the frame are shown in Figure 2.1. The moments at all the salient points are computed by slope deflection method and are shown in Figure 2.2. Plastic moment capacities of different elements of the frame are obtained by multiplying the maximum working load moments in the elements by the corresponding load factors. The plastic moment capacities are shown in Figure 2.3. The extra reinforcement at the joint is assumed to be extended into the beam and columns for a portion of one tenth of the length of member

Let P_w = working load on the frame

P_u = ultimate load on the frame

F_b = beam load factor

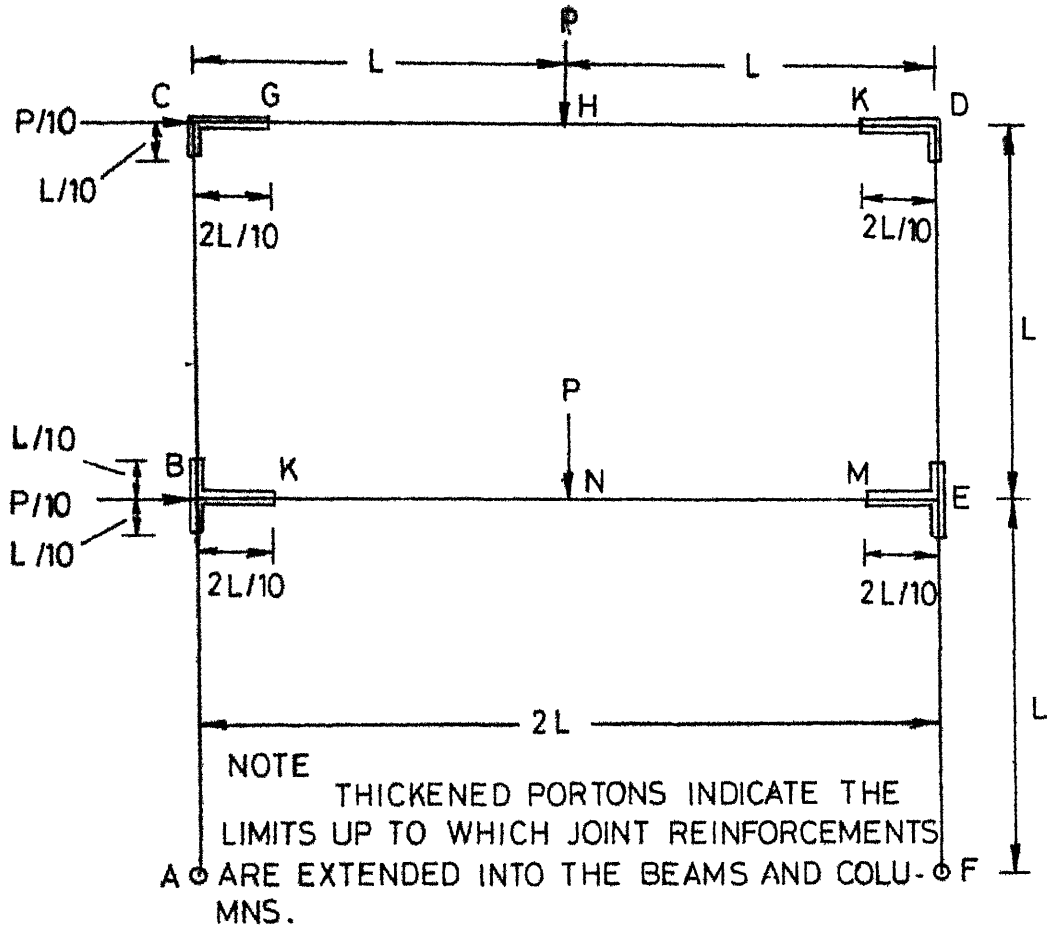

FIG 2.1 GENERAL DIMENSIONS OF FRAME.

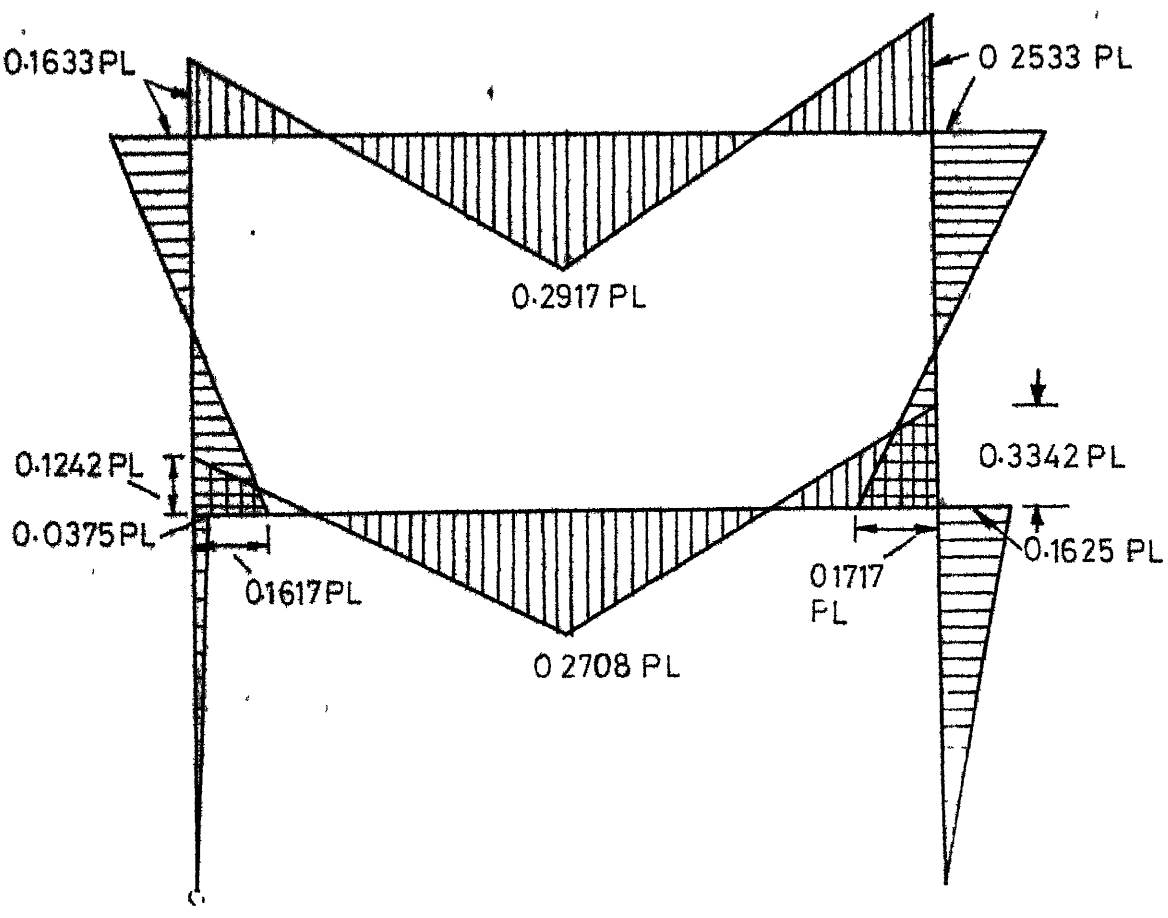

FIG 2.2 BENDING MOMENT DIAGRAM

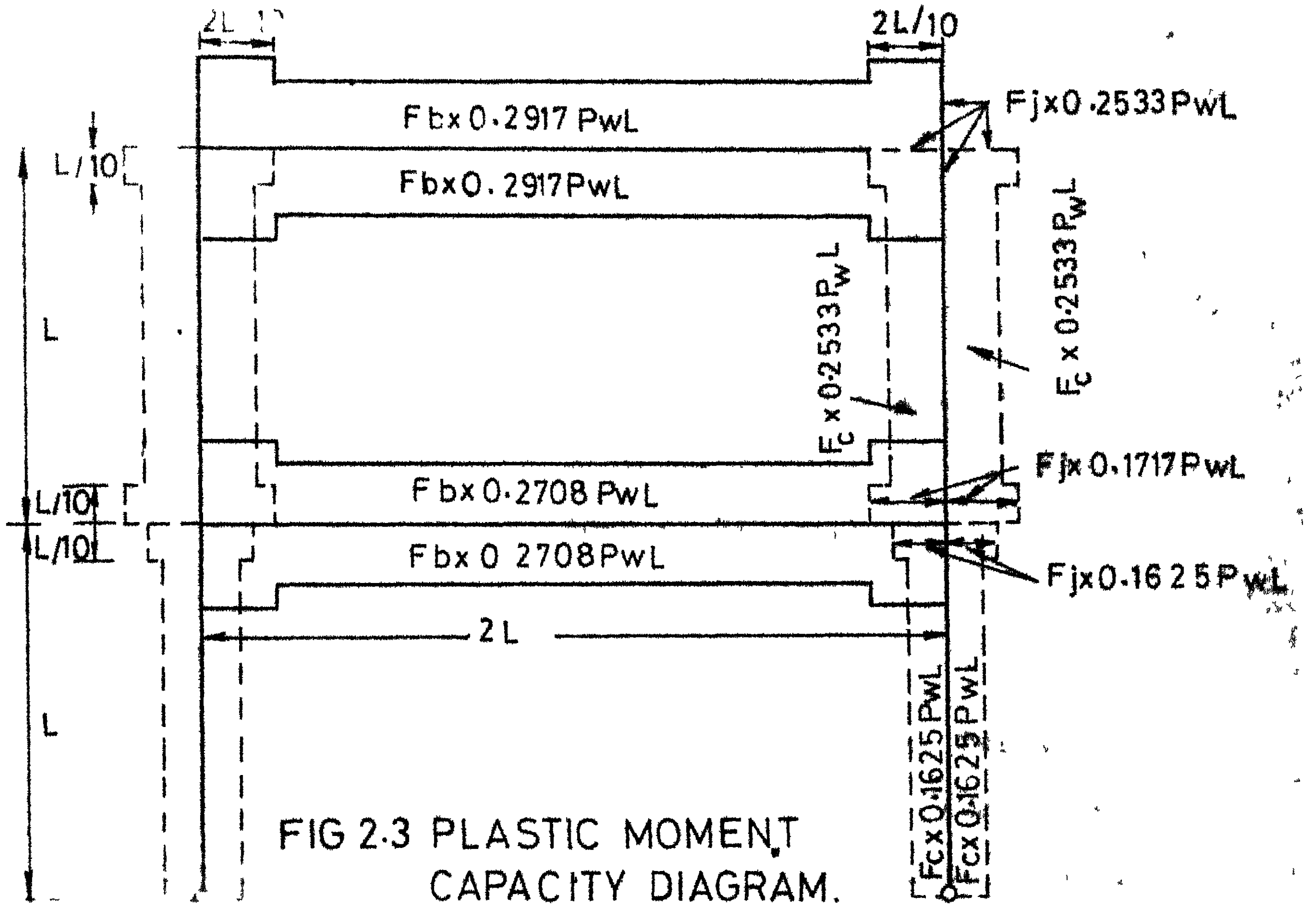

FIG 2·3 PLASTIC MOMENT CAPACITY DIAGRAM.

F_j = joint load factor

F_o = overall load factor

$(M_{pb})^u$, $(M_{pb})^l$ = plastic moment capacities of upper and lower beams

$(M_{pj})^u$, $(M_{pj})^l$ = plastic moment capacities of upper and lower joints.

M_{wn}, M_{wm}, M_{we} etc., = working load moments at the points N,M,E etc. of the frame shown in Fig. 2.1

Programme 1 :

As the load is increased gradually, the bending moments at the centres of upper and lower beams reach the moment capacities at the sections and the first two plastic hinges are formed at H & N. As the load is further increased the redistribution of moments take place till the moments at the joints reach their moment capacities consequently forming the next two hinges at D & E. The redistribution of moments and the progressive formation of plastic hinges in upper and lower beams are indicated

in Fig. 2.4(a) to 2.4(c). The collapse mechanism-1, after the formation of plastic hinges at H, N, D & E is shown in Fig. 2.4 (d). From the mechanism diagram, and using the principle of virtual work, the expression for F_o in terms of F_b and F_j is obtained as follows:

Let U : work done by the external forces

V : energy absorbed in the rotation of plastic hinges.

$$U = \frac{P_u}{10} (L\theta + 2 L\theta) + P_u (L\theta + L\theta)$$

$$= 2.3 P_u L\theta \tag{1}$$

$$V = F_b . (0.2917 P_w L).2 \theta + F_b .(0.2708 P_w L) 2\theta$$

$$+F_j (0.2533 P_w L) 2 \theta + F_j (0.3342 P_w L) 2\theta$$

$$= (1.125 F_b + 1.175 F_j) P_w L\theta \tag{2}$$

Equating the expressions for U and V and substituting F_o for P_u/P_w,

$$F_o = 0.49 F_b + 0.51 F_j \tag{3}$$

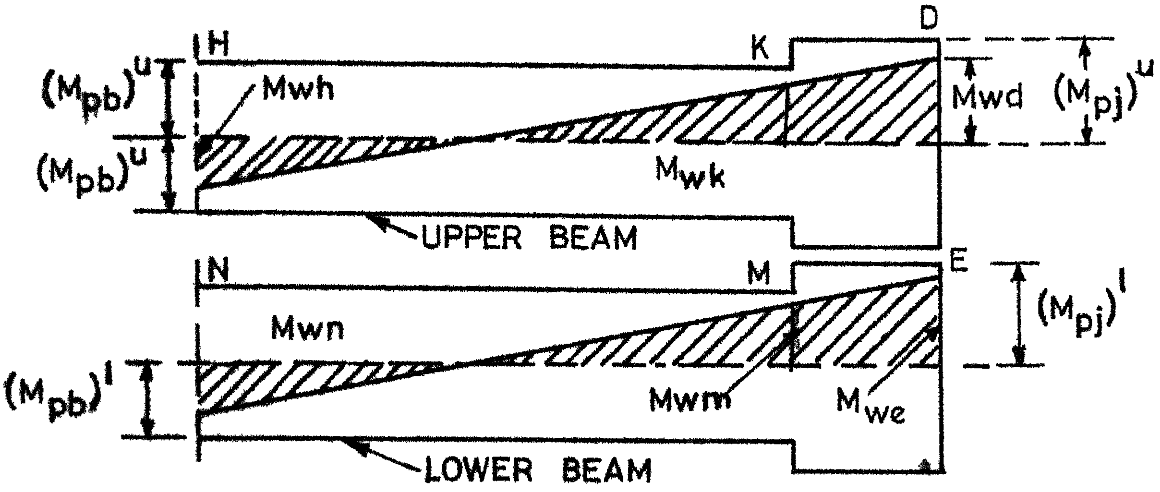

FIG. 2.4 (a) MOMENTS AT WORKING LOADS

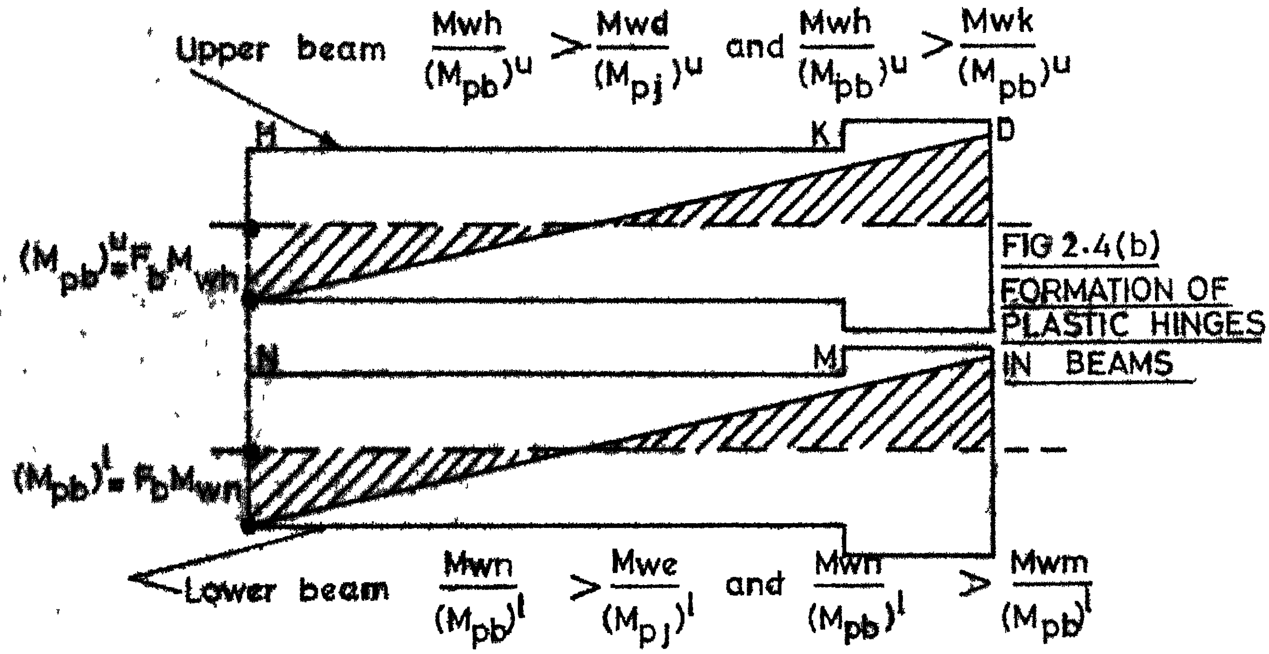

FIG 2.4 (b) FORMATION OF PLASTIC HINGES IN BEAMS

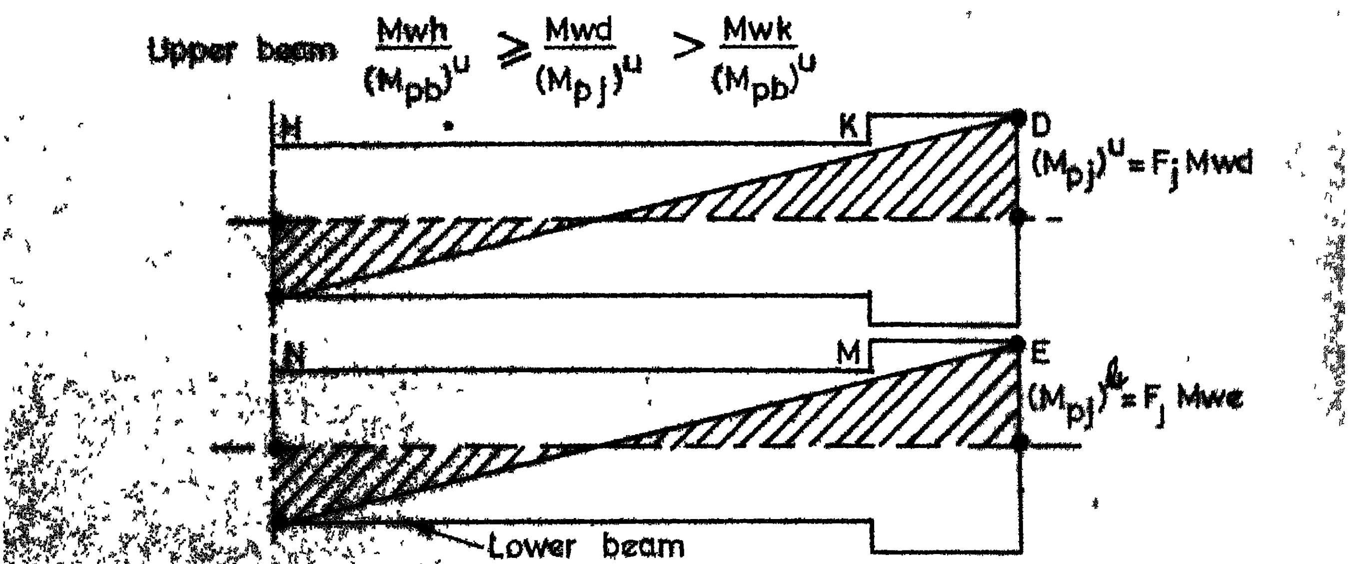

FIG. 2.4 (c) HINGE FORMATION AT JOINTS

FIG. 2.4 PROGRESSIVE HINGE FORMATION WHEN $F_j < 1.22 F_b$

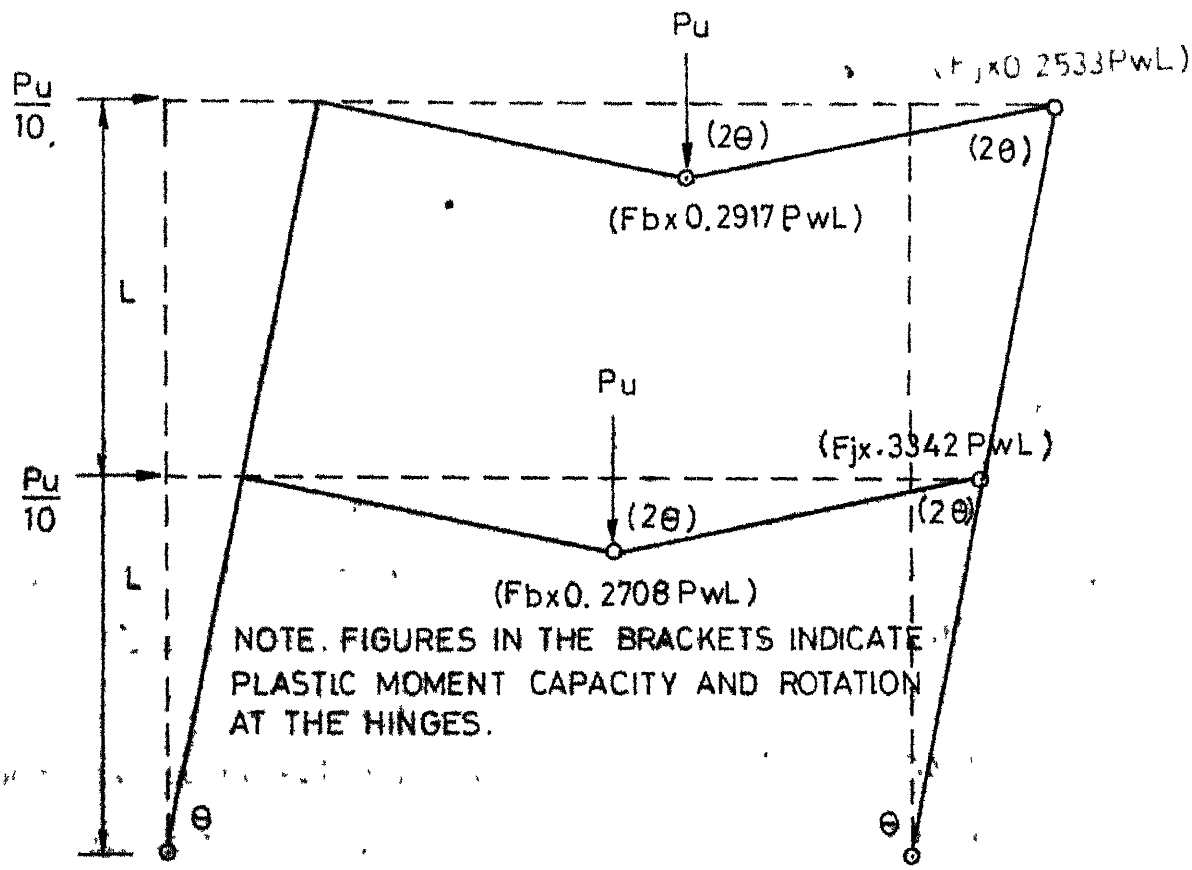

FIG. 2-4(d) COLLAPSE MECHANISM-1

Programme 2:

While keeping the moment capacities of the beam portions unchanged the moment capacities of the joint portions are increased by increasing the joint load factor. The redistribution of moments, position of plastic hinges and failure mechanism is found to be same as in programme 1, until a particular value of F_j. For this value of F_j, the bending moments at E & M, i.e. at the centre of the joint and at the end of the joint reinforcement of lower beam have reached the moment capacities of the sections symultaneously during the redistribution of moments. Position of hinges in the upper beam is the same as in programme 1. This formation is shown in Fig. 2.5 (a). The values of the bending moments and moment capacities at this stage in the lower beam in terms of F_b,F_j,P_w etc., are shown in Fig. 2.5 (b).

From Fig. 2.5 (b) it is clear that the simultaneous formation of hinges at E and M take place when

$$F_j(0.3342\ P_wL) = F_b(0.2708\ P_wL) + \frac{1}{4}\ F_b(0.5416\ P_wL)$$

$$\text{or when } F_j = 1.22\ F_b \qquad (4)$$

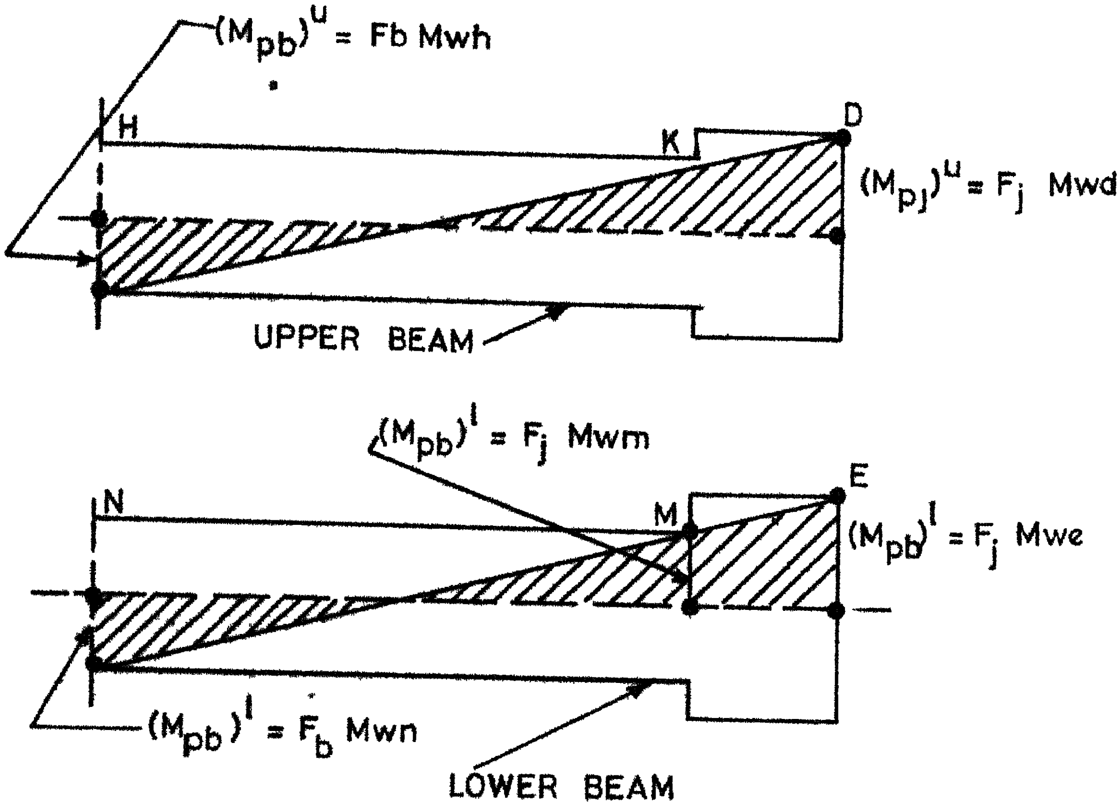

FIG 2 5 (a) POSITION OF PLASTIC HINGES WHEN $F_j = 1.22\ F_b$

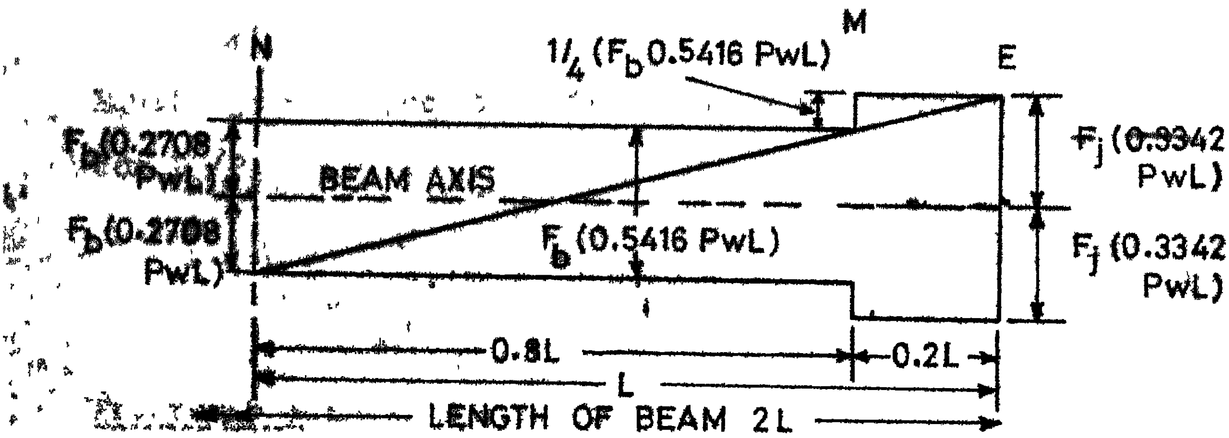

FIG 2.5(b) DETAILS OF MOMENTS IN LOWER BEAM BEFORE COLLAPSE WHEN $F_j = 1.22\ F_b$

FIG.2.5 HINGE FORMATION WHEN $F_j = 1.22\ F_b$

Therefore when F_j is less than 1.22 F_b, the formation of hinges is as shown in Figures 2.4 (a) to 2.4(c) in programme 1 and when it is equal to 1.22 F_b, it is as shown in Fig. 2.5 (a). From the figure, it is evident that when F_j is just greater than 1.22 F_b, the hinge at the point of the lower beam forms only at M and not at E. The failure mechanism-2, when F_j is greater than 1.22 F_b is shown in Fig. 2.6. From this diagram,

$$U = 2.3\ P_u L\theta \quad \text{(same as in programme 1)}$$

$$V = F_b(0.2917\ P_w L)2\theta + F_b(0.2708\ P_w L)\ (2.5\theta + 2.5\theta)$$

$$+ F_j\ (0.2533\ P_w L)2\theta$$

$$= \quad (1.9374\ F_b + 0.5066\ F_j)\ P_w L\theta \tag{5}$$

Equating the expressions for U and V and substituting F_o for P_u/P_w

$$F_o = 0.842\ F_b + 0.22\ F_j \tag{6}$$

Programme 3 :

As the value of F_j is further increased at another

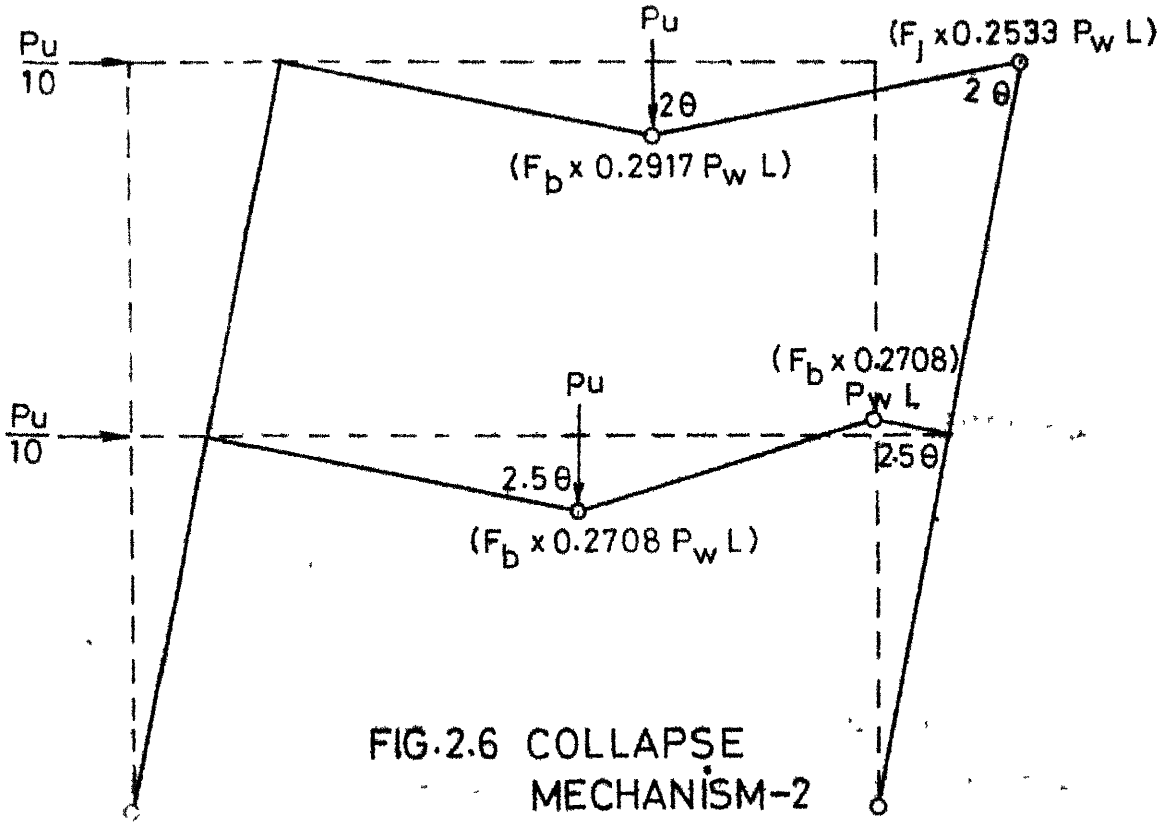

FIG. 2.6 COLLAPSE MECHANISM-2

stage, in exactly similar manner described in programme 2, simultaneous hinge formations at K & D take place in the upper beam. The position of hinge formations in upper and lower beam at this stage is shown in Fig. 2.7(a).
In Fig. 2.7 (b) details of bending moments and moment capacities in the upper beam at this stage are given in terms of F_o, F_j and P_w.

From Fig. 2.7 (b), it is clear that the simultaneous hinge formations at K and D take place when

$$F_j\ (0.2533\ P_wL) = F_b(0.2917\ P_wL) + (1/4)\,F_b(0.5834\ P_wL)$$

$$\text{or when}\quad F_j = 1.73\ F_b \qquad (7)$$

It is evident that when F_j is just greater than 1.73 F_b, the hinge will be formed only at K and not at D. The failure mechanism - 3, when F_j is greater than 1.73 F_b is shown in Fig. 2.8. From this diagram,

$$U = 2.3\ P_uL\theta \text{ (same as in programme 1)}$$

$$V = 2 \times F_b(0.2917\ P_wL)\ 2.5\theta + 2\ F_b(0.2708\ P_wL)2.5\theta$$

$$= F_b \cdot 2.82\ P_wL\theta \qquad (8)$$

Equating the expressions for U and V and substituting F_o for P_u/P_w

$$F_o = 1.225\ F_b \tag{9}$$

This expression for overall load factor shows that beyond the value of F_j equal to 1.73 F_b, the effect of increase in the joint load factor is not felt in the overall load factor.

Summing up the different stages considered above, the variation of overall load factor with the increase in joint load factor, for a single bay two storeyed frame under the pattern of loading shown in Fig. 2.1 is given by the following expressions.

i) $F_o = 0.49\ F_b + 0.51\ F_j \quad \text{when } F_j \leq 1.22\ F_b \quad (10)$

ii) $F_o = 0.842 F_b + 0.22\ F_j \quad \text{when } 1.22\ F_b \leq F_j \leq 1.73 F_b \quad (11)$

iii) $F_o = 1.225\ F_b \quad \text{when } F_j \geq 1.73\ F_b \quad (12)$

The above results are represented graphically in Fig. 2.9 as F_o versus F_j for a constant value of F_b.

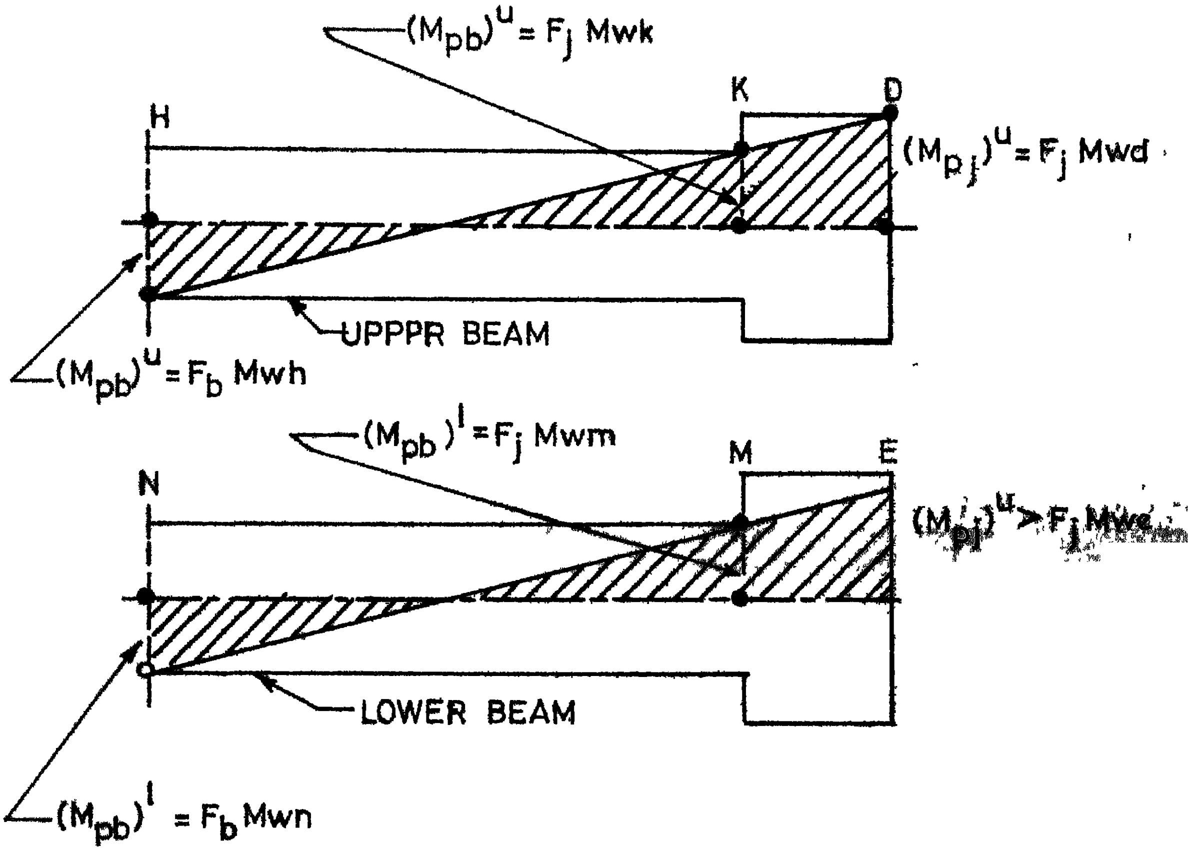

FIG. 2.7 (a) POSITION OF PLASTIC HINGES WHEN $F_j = 1.73\ F_b$

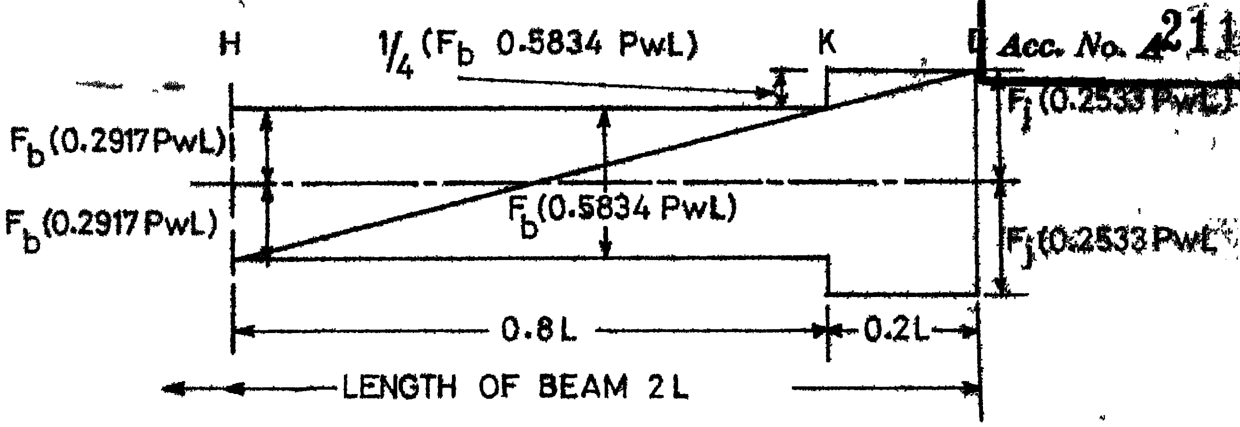

FIG 2.7 (b) DETAILS OF MOMENTS IN UPPER BEAM BEFORE COLLAPSE WHEN $F_j = 1.73\ F_b$

FIG. 2.7 HINGE FORMATION WHEN $F_j = 1.73\ F_b$

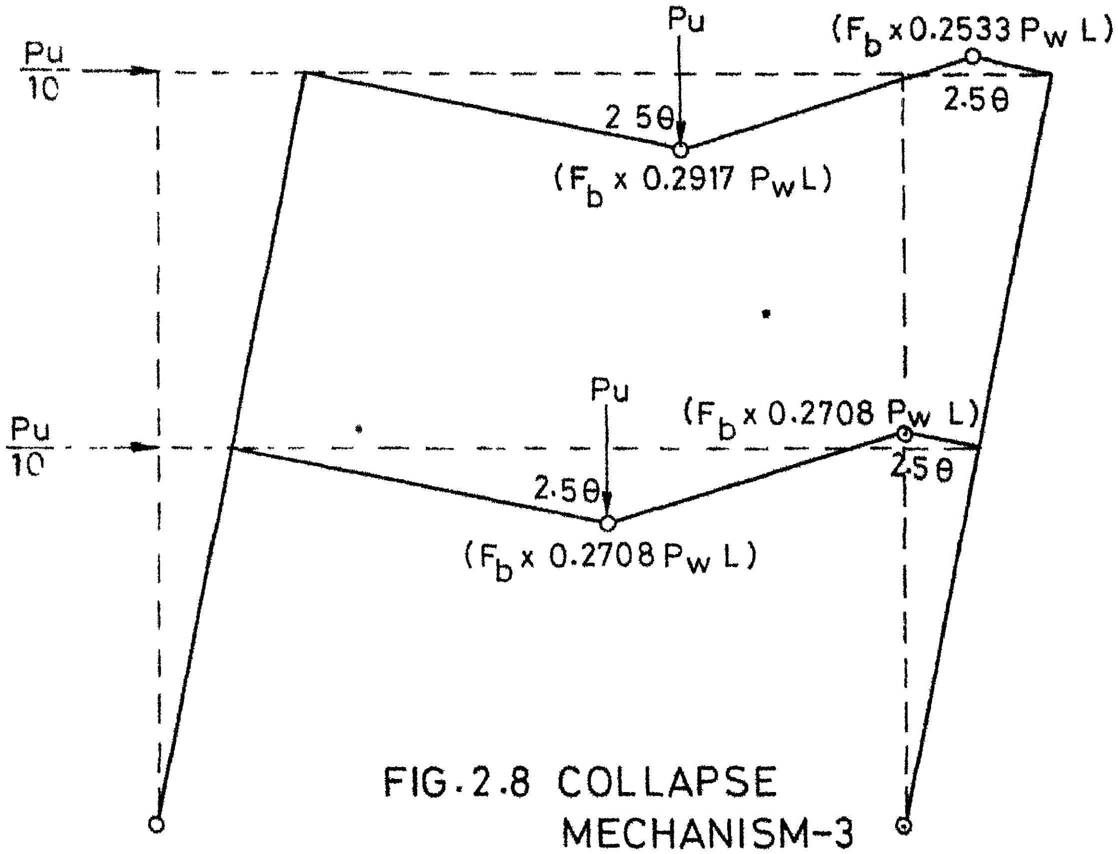

FIG. 2.8 COLLAPSE MECHANISM-3

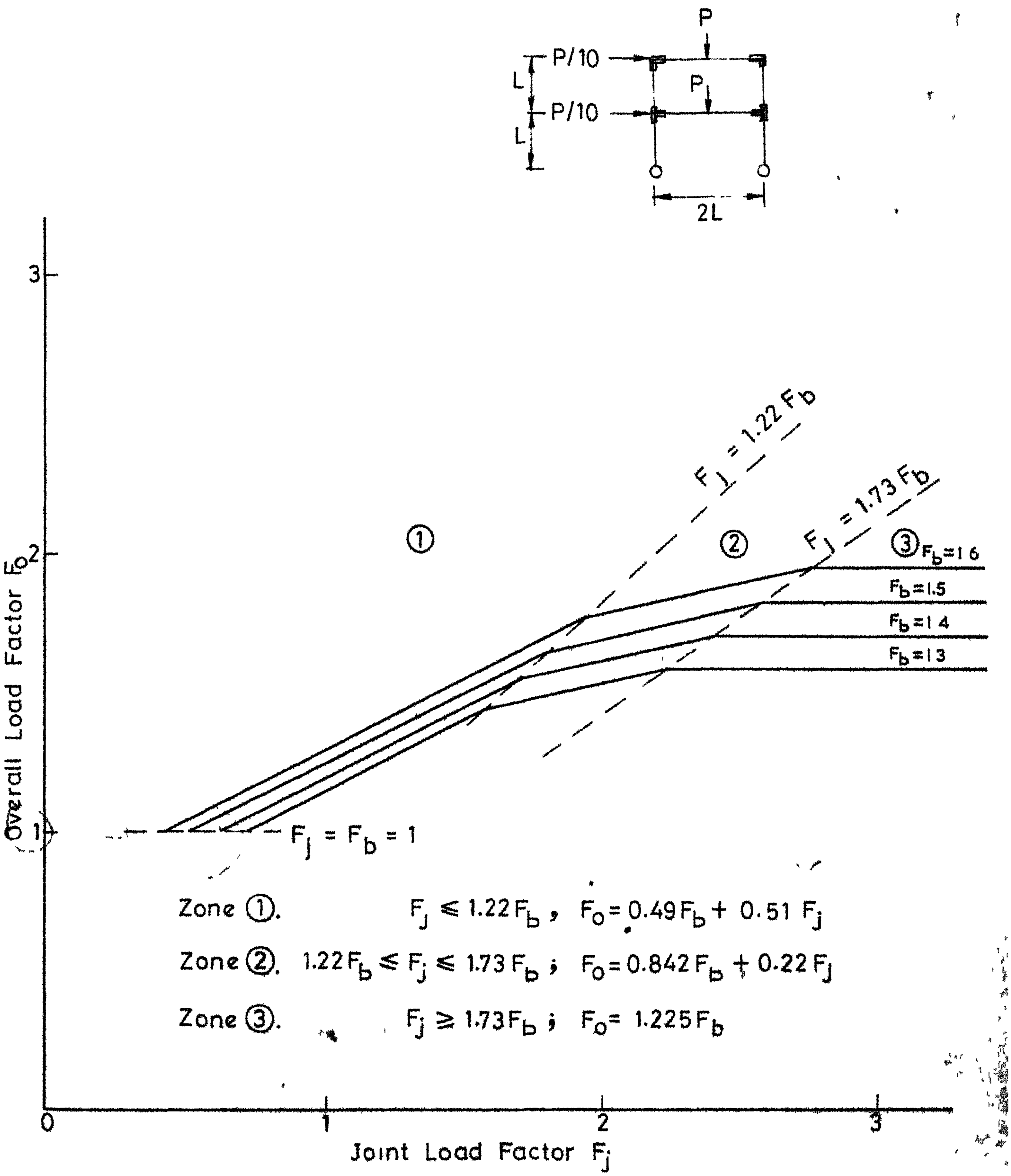

Fig.2.9 Influence of F_j on F_o in a single bay two storey frame

The graphs are also drawn for different constant values of F_b equal to 1.4, 1.5 and 1.6 to get an idea as to how the overall load factor behaves with the increase in beam load factor also.

Similar analysis is made for single bay, three storeyed, four storeyed and single storeyed portal frames and the results are diagramatically represented in Figs. 2.10, 2.11 & 2.12 respectively. The following observations are made from the results:

1. Overall load factor increases with the joint load factor upto a certain value of the latter beyond which it remains constant.
2. Rate of increase in F_o is more in the initial stages compared to that in later stages.
3. As the order of indeterminacy of the structure increases the constant value of F_o is reached at lower levels of F_j.
4. The F_o VS F_j curve is very similar to the piecewise linear elasto-plastic deformations curve.

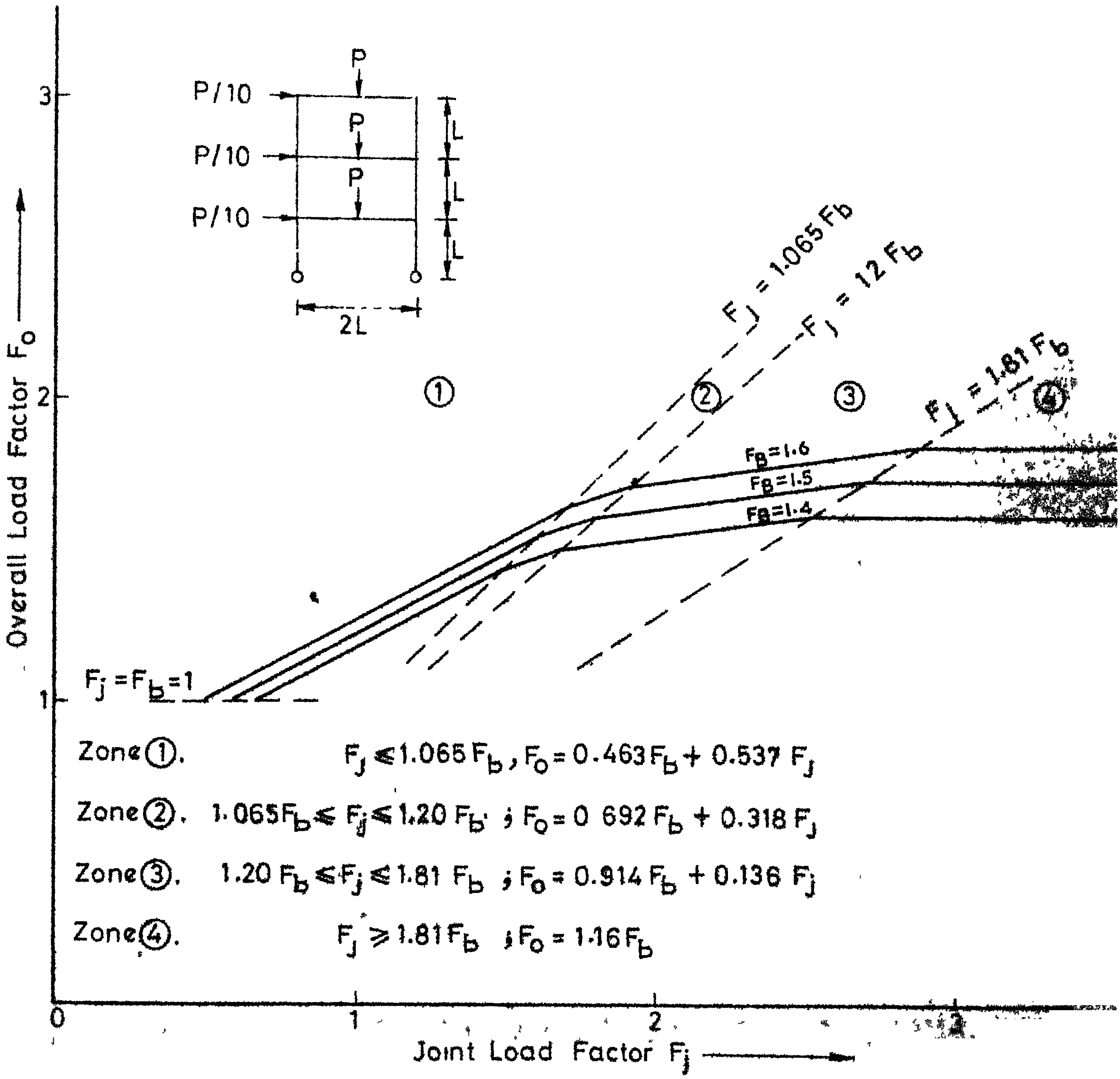

Fig. 2.10 Influence of F_j on F_o in a single bay three storey [redacted]

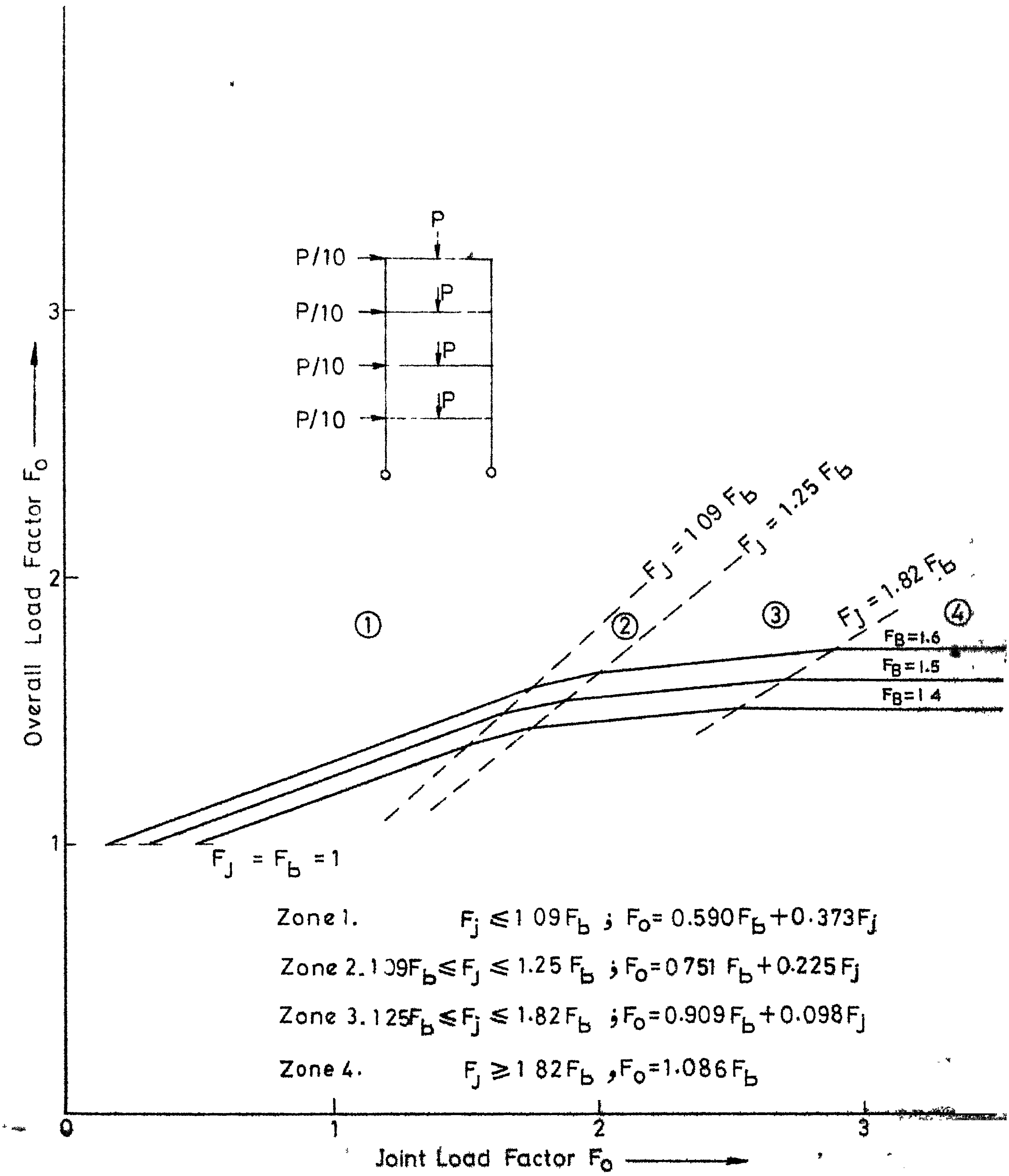

Fig. 2·11 Influence of F_j on F_o in a single bay four storey frame

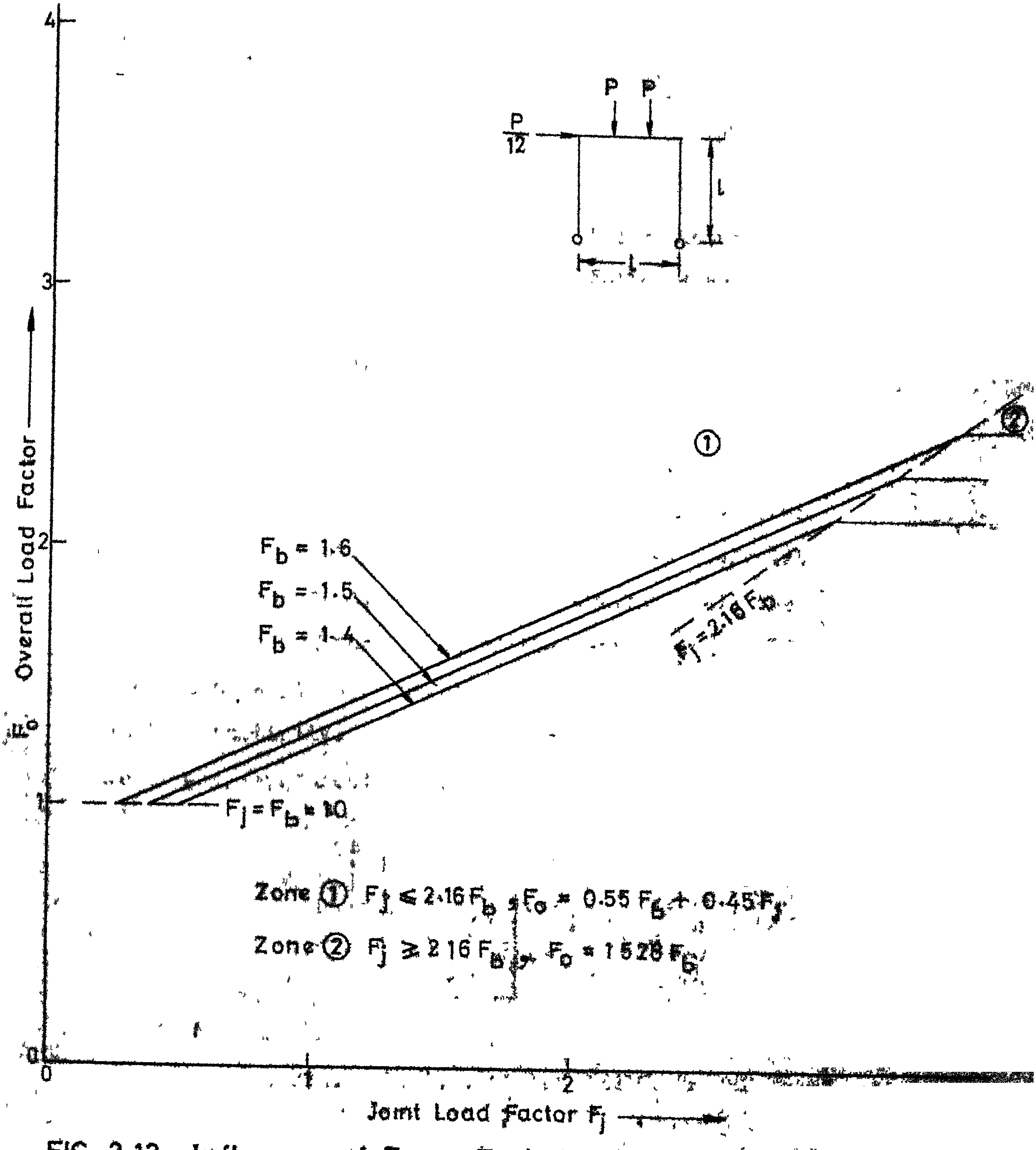

FIG. 2.12 Influence of F_j on F_o in a single bay single storey frame

2.5 Cost analysis :

Increase in the strength of the structure with the increase in joint load factor is followed by increase in the cost of structure also due to the extra reinforcement provided at the joints. An approximate cost analysis is therefore made to have an estimate of the increase in the cost of structure with the increase in joint load factors. The same two storeyed frame shown in Fig. 2.1 is considered for a typical cost analysis. For the purpose of approximate cost analysis, all the members including the joints of the frame are assumed to have the same plastic moment capacity throughout their lengths, initially. It is also assumed that all the members are singly reinforced.

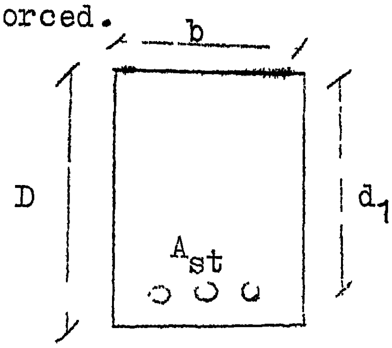

FIG. 2.13 C.S. OF THE MEMBER OF THE FRAME

Let f_{sy} = yield stress in steel, assumed as 3000 kg/cm^2

f_{cu} = cube strength of concrete assumed as 250 kg/cm^2

p = ratio of steel = A_{st}/bd_1

d_1 = effective depth of section = $\frac{D}{1.1}$

C_c = cost of concrete per unit volume

C_s = cost of steel per unit volume = 60 C_c, say.

The governing equation for a singly reinforced section is given (10) as

$$m = p\left[1 - \frac{1}{1.1}\ \frac{f_{sy}}{f_{cu}}\right] \tag{13}$$

where the non-dimensional moment parameter 'm' is given by

$$m = \frac{M_u}{bd_1^2\, f_{sy}} \tag{14}$$

The relation between m and p is given (10) in the shape of a graph The initial value of steel ratio p_1 is assumed as 0.01 and the corresponding value of m_1 is obtained from the graph.

Total length of all members in the frame = 8L

Length of all the portions where additional reinforcement is to be provided near the joints

$$= 4 \times \frac{2L}{10} + \frac{6xL}{10} = 1.4\ L$$

Cost of concrete in the entire structure $= bD8LC_c$

$$= 8.8\ bd_1 L\ C_c \qquad (15)$$

Area of steel in the section initially $= p_i\ bd_1$

Cost of steel in the entire structure (initially)

$$= p_i\ bd_1\ (8L)\ 60\ C_c \qquad (16)$$

$\therefore$ Initial cost of the structure. $= bd_1\ L\ C_c(8.8+480\ p_i)$ (17)

Let there be an increase of x percent in the joint load factor. Then there will be a corresponding increase in the plastic moment capacity of the section at the joint and consequently an increase in the initial moment parameter

$$m = m_i\ \left(1 + \frac{x}{100} \right) \qquad (18)$$

Length of all the portions where additional reinforcement is to be provided near the joints

$$= 4 \times \frac{2L}{10} + \frac{6 \times L}{10} = 1.4\ L$$

Cost of concrete in the entire structure $= bD8LC_c$

$$= 8.8\ bd_1 L\ C_c \qquad (15)$$

Area of steel in the section initially $= p_i\ bd_1$

Cost of steel in the entire structure (initially)

$$= p_i\ bd_1\ (8L)\ 60\ C_c \qquad (16)$$

$\therefore$ Initial cost of the structure $= bd_1\ L\ C_c(8.8+480\ p_i)$ (17)

Let there be an increase of x percent in the joint load factor. Then there will be a corresponding increase in the plastic moment capacity of the section at the joint and consequently an increase in the initial moment parameter

$$m = m_i\ \left(1 + \frac{x}{100} \right) \qquad (18)$$

The increased steel ratio corresponding to the increased value of m is obtained from the graph. If the increase in the ratio of steel is 'q' then $p = p_1 + q$.

Additional area of steel $= (p-p_1)\ b\ d_1 = q\ bd_1$ (19)

Additional cost of the frame due to the increase in the area of steel

$$= qb\ d_1 \times 1.4\ L \times 60\ C_c$$

Percentage increase in the total cost of the frame

$$= \frac{\text{increase in cost}}{\text{initial cost}} \times 100$$

$$= \frac{84\ q\ b\ d_1\ L\ C_c}{(8.8 + 480\ p_1)\ b\ d_1\ L\ C_c} \times 100$$

Substituting $p_1 = 0.01$

$$= 617\ q \tag{20}$$

where q is the increase in the ratio of steel obtained from the graph.

The percentage increase in the total cost is calculated for different percentage increases in the joint load factor and the particulars are tabulated in Table 2.3

TABLE 2.3 Cost particulars of two storeyed frame

Percentage increase in F_J x %	Moment parameter from eqn 18 m	Steel ratio from graph p	Increase in Steel ratio $q=p-p_1$	Percentage increase in total cost from eqn.(20)
0	0.0088	0.0100	0	0
20	0.0106	0.0125	0.0025	1.53
40	0.0123	0.0150	0.0050	3.07
60	0.0141	0.0178	0.0078	4.73
80	0.0158	0.0203	0.0103	6.35
100	0.0176	0.0225	0.0125	7.72
120	0.0194	0.0248	0.0148	9.13
140	0.0211	0.0270	0.0170	10.50

Figure 2.14 effectively brings out the percentage increases in the cost of the structure and the strength of the structure compared to the increase in joint strength. The difference in the ordinates of these two plots is shown as Marginal increase between strength and cost. Since the cost and strength are two dissimilar quantities, the peak of this curve does not represent any optimal condition in the design. However it makes clear that the cost of structure continues to increase with the increase in joint load factor while the strength remains constant beyond certain value of F_J.

Similar cost analysis is made with reference to the three storeyed, four storeyed and single storeyed frames and the results are diagramatically represented in Figures 2.15, 2.16 and 2.17 respectively.

Limitation of the above analysis :

The formation of collapse mechanisms in the above analysis is based on the assumption that the plastic hinges formed at earlier stages have enough rotational capacities and that they do not fail before the hinges of the later stages are formed. This assumption may not be justified

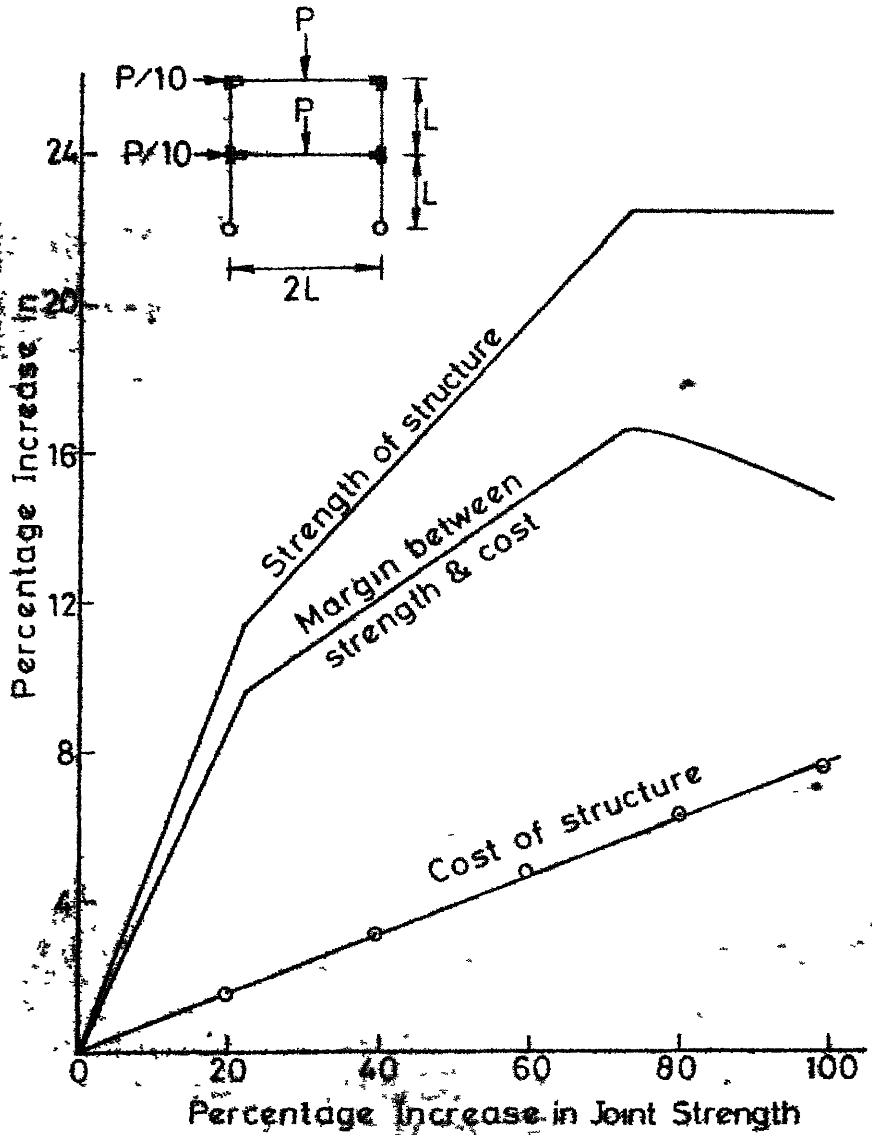

Fig. 2·14 Strength & cost particulars of two storeyed frame

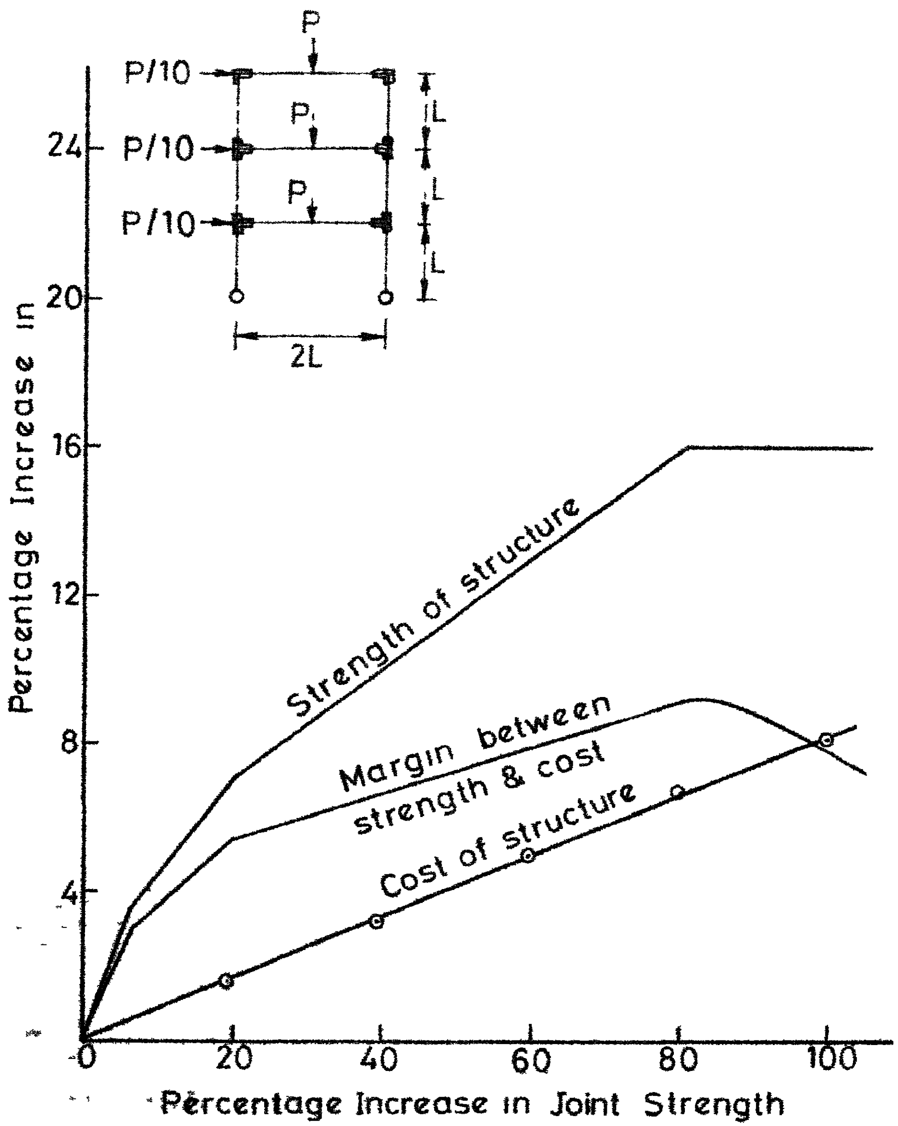

Fig. 2·15 Strength & cost particulars of three storeyed frame

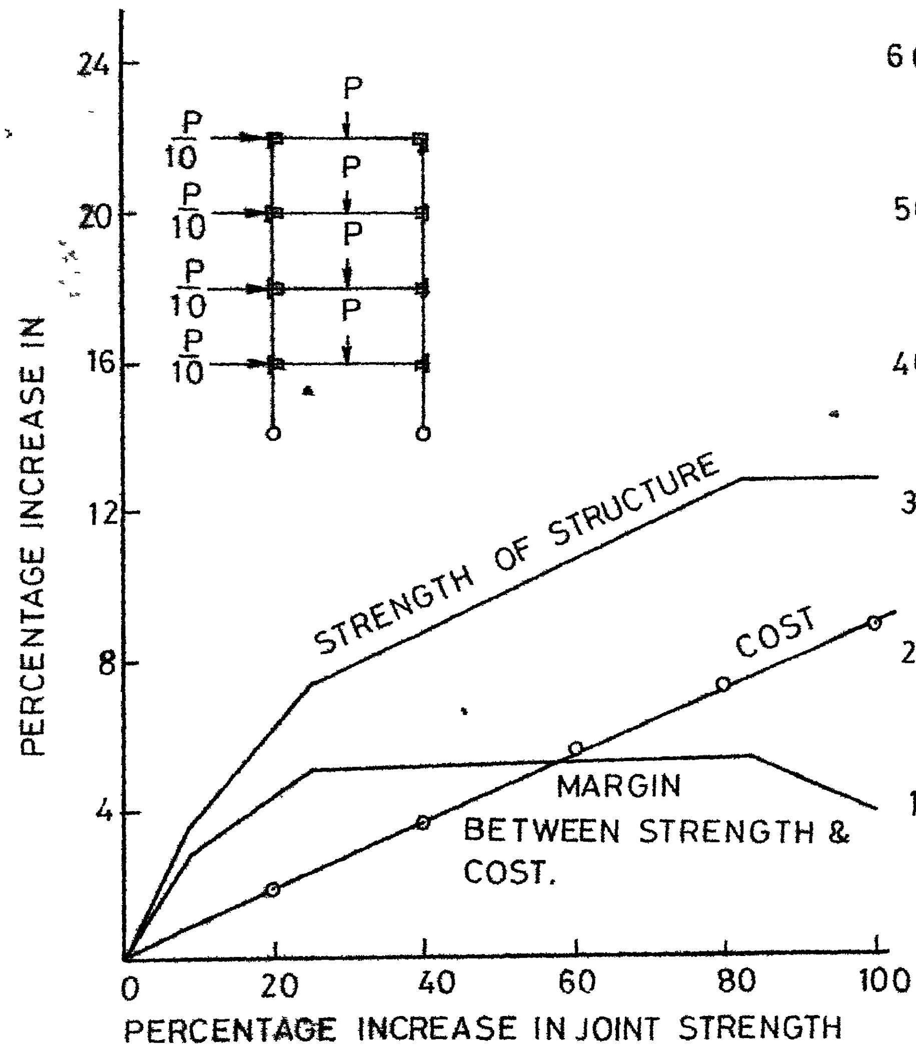

FIG 2.16 STRENGTH & COST PARTICULARS OF FOUR STOREYED FRAME.

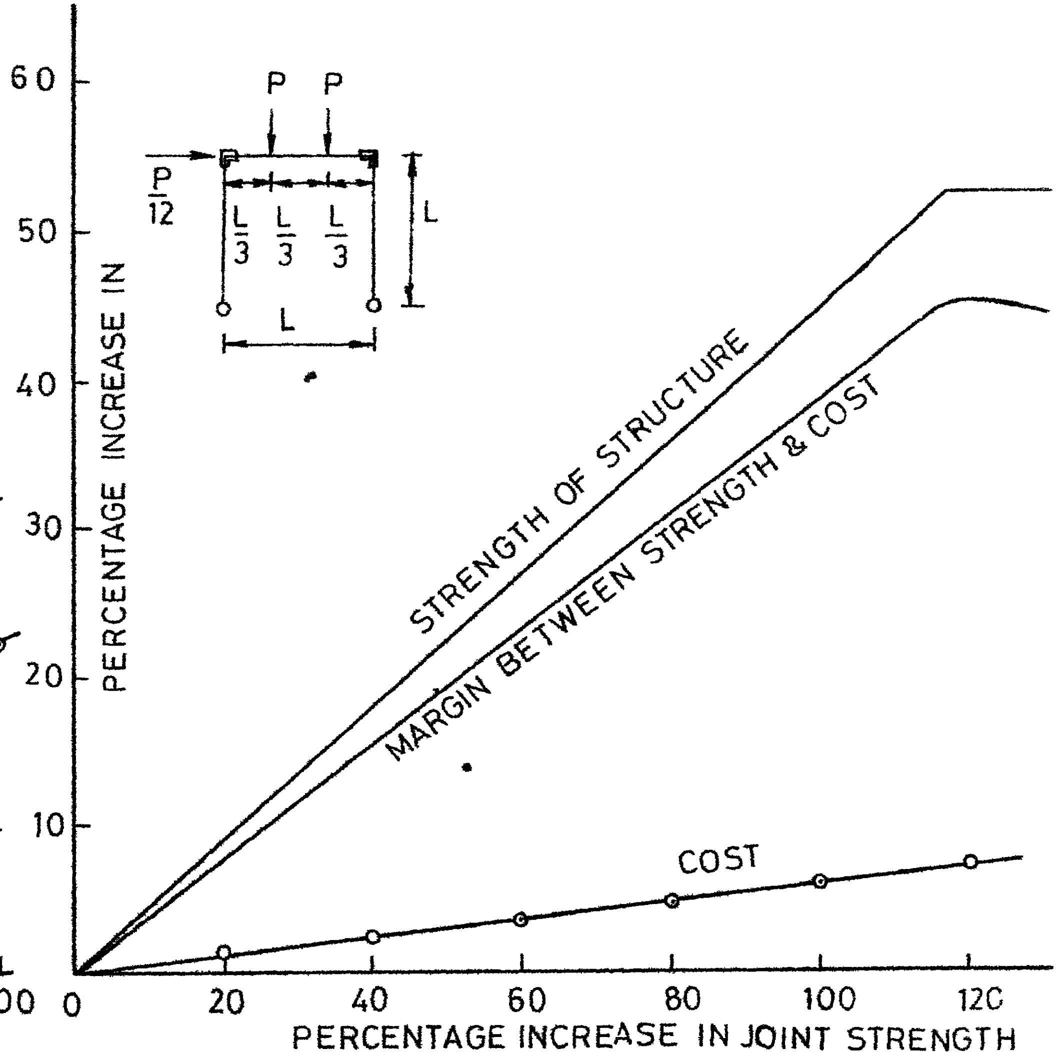

FIG 2.17 STRENGTH & COST PARTICULARS OF SINGLE STOREYED FRAME

when the margin between the formation of earlier and later hinges is considerable, because of the limited ultimate strain capacity of a R.C. section. Therefore the rotational capacities of the different critical sections are to be carefully assessed before taking it for granted that the increase in strength of structure represented by the increase in the value of the overall load factor is valid.

CHAPTER III

EXPERIMENTAL INVESTIGATION

3.1 Test specimens :

Object

As mentioned in Chapter 1, the object of experimental investigation was to study the behaviour of prestressed precast composite portal frame under the action of both vertical and horizontal loads such that the load pulsates several lakhs of times repeating between simulated dead load and working load levels and with occasional peak loads.

Selection of specimen

A single bay single storey portal frame with hinged supports was selected for testing purposes. The dimensions of the specimen were chosen (i) to facilitate easy handling (ii) to suit the capacities of the hydraulic jacks available in the laboratory for loading purposes and (iii) to represent to a moderate scale the actual dimensions of the portal frames. The centre line dimensions and the loading positions

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of the frame are shown in Figure 3.1. 10 cm x 15 cm cross sectional dimensions were chosen for both column and beam sections through out the frame. The portal frame was assembled out of two column elements of reinforced concrete and a beam element of prestressed concrete. An unbonded post-tensioned beam was adopted by embedding the tendons inserted through thin polythene tubes in the specimen. Secondary mild steel reinforcement was also used in the prestressed concrete beam section.

Design of specimen

The specimen was designed by load factor method giving different load factors to different elements. A load factor of 1.7 for columns and 1.5 for beams was adopted uniformly for all specimens. For joints, the load factor was varied from 1.8 to 2.7. The specimen was analysed for two vertical loads 'P' each at one-third points on the beam and a horizontal load KP at the right hand top corner of the frame. The ultimate moment capacities, for which the beam, column and joint sections were designed were obtained by multiplying the corresponding maximum working load moments with the corresponding chosen load factors. Assuming that the sections had sufficient

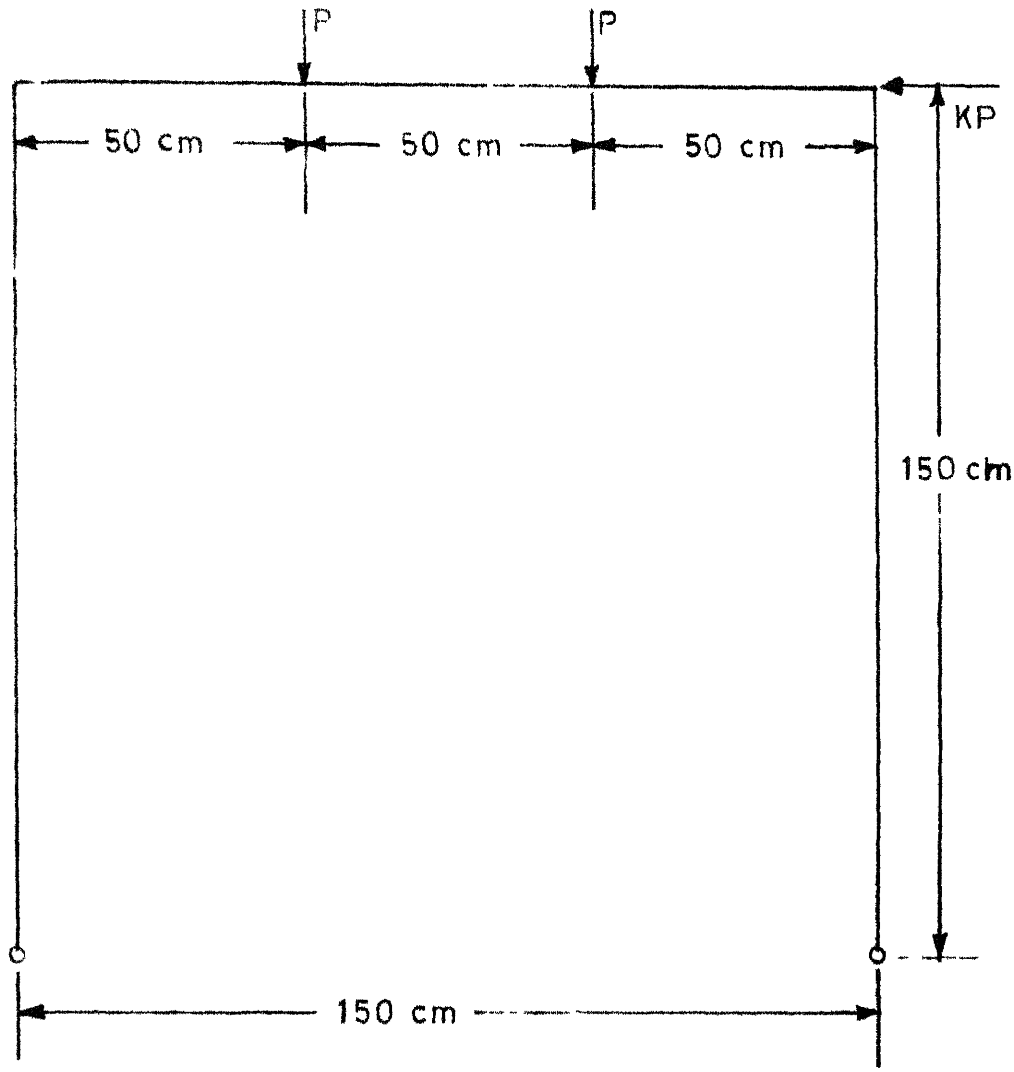


FIG. 3.1 CENTRE LINE DIMENSIONS AND LOADING POSITIONS OF THE FRAME SPECIMEN

rotation capacity to allow the full redistribution of moments, the ultimate load P_u on the frame was determined by the collapse mechanism using the ultimate moments of the sections. The ratio of ultimate load P_u and the working load P_w gives the overall load factor for the specimen.

The reinforcement at different sections in the frame was provided to obtain the desired ultimate moment capacities. The strength required for the joints to give the increased joint load factor was achieved by providing necessary amount of extra mild steel reinforcement at the joint. This extra reinforcement was extended into the beam and column for a sufficient length to satisfy the bond requirements. Beams were provided with necessary shear reinforcement, with suitable factor of safety to avoid the shear failure in the beam.

Detailed design calculations for specimen No.1 are given in Appendix-A. The reinforcement details and other dimensions are given in Figs. 3.2 and 3.3, and in Table 3.1.

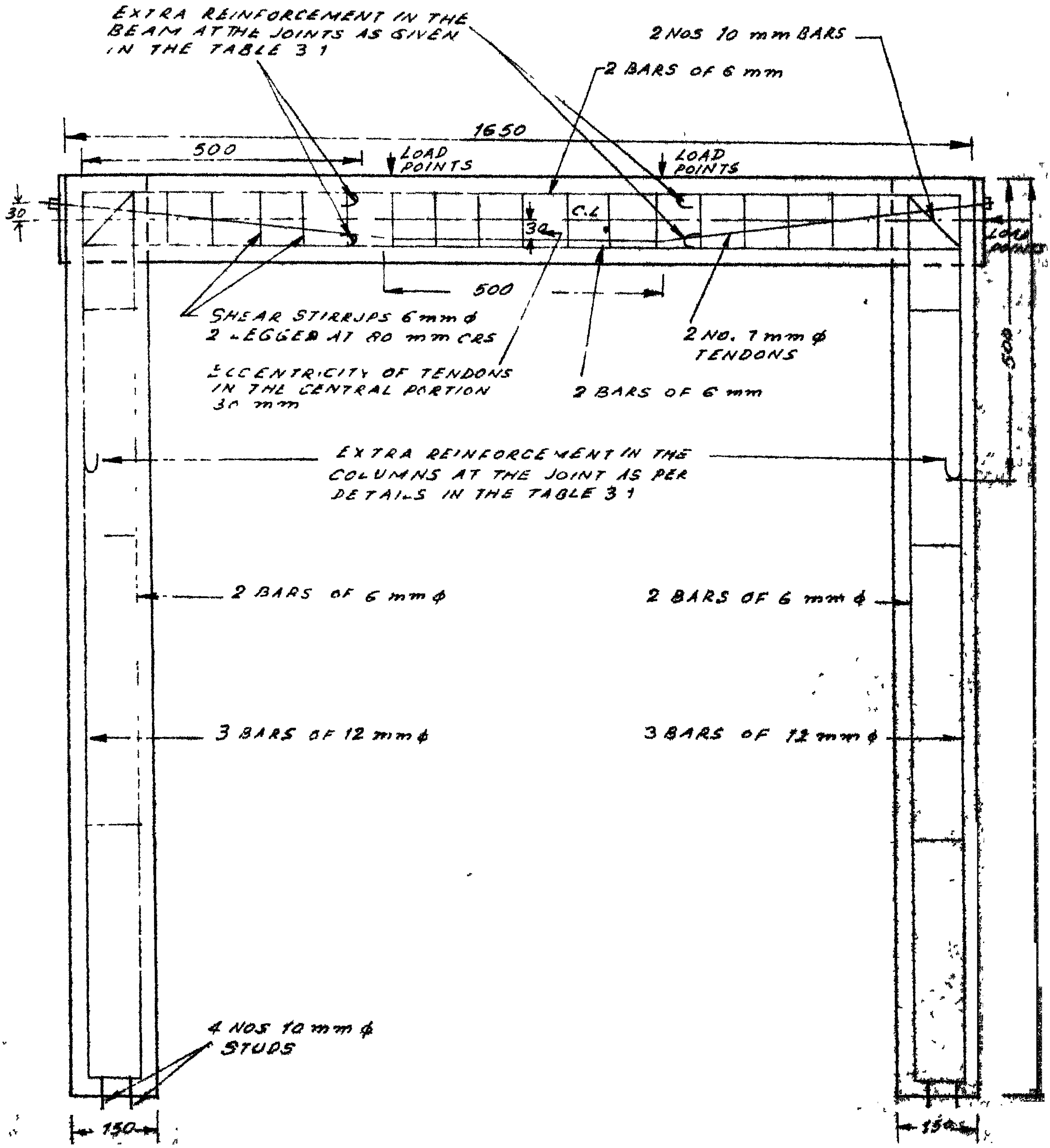
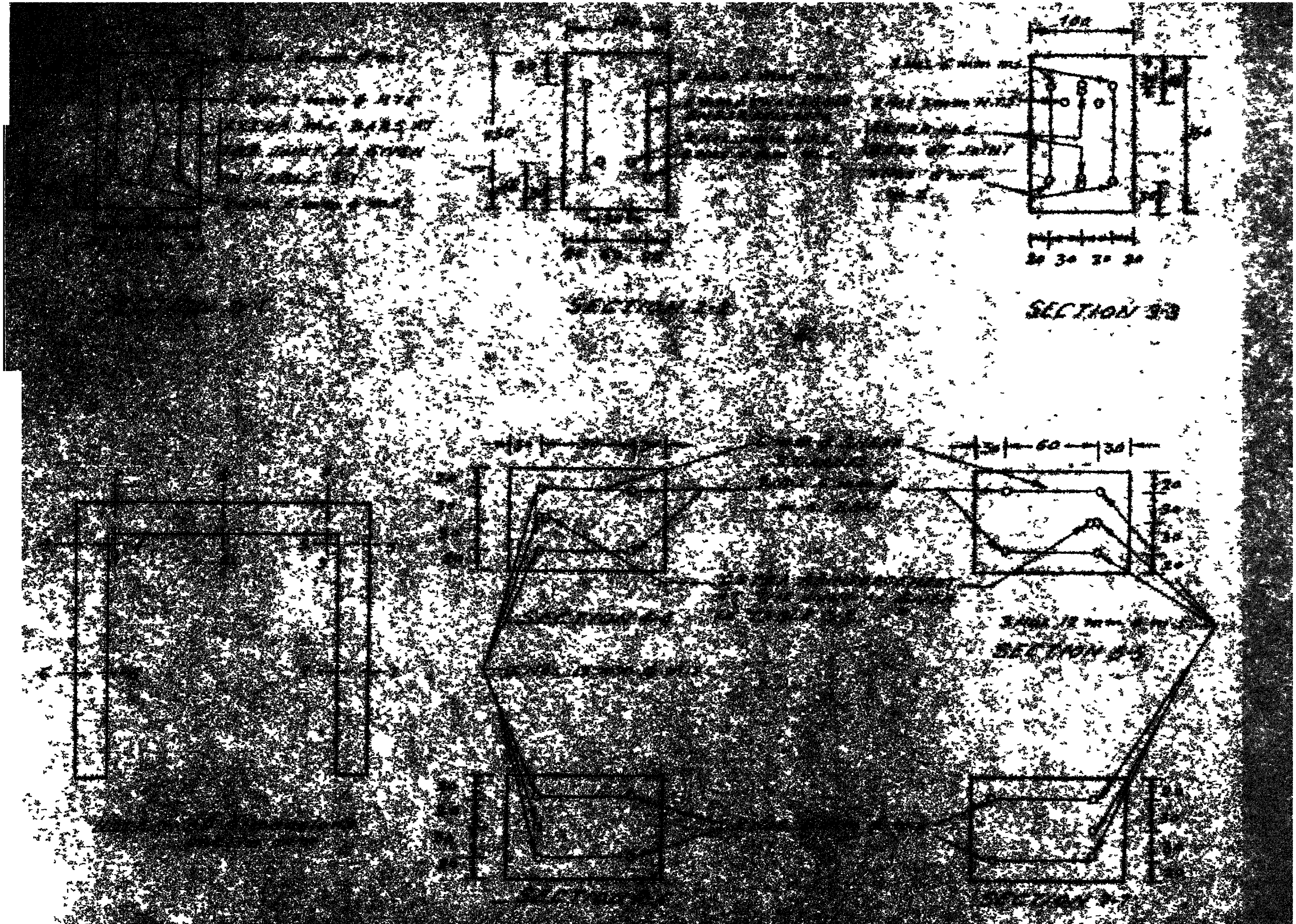


FIG. 32 : GENERAL ARRANGEMENT OF REINFORCEMENT
(All dimensions in mm)



[illegible] OF THE FRAME

TABLE 3.1

DETAILS OF REINFORCEMENTS OF THE JOINTS

JOINT ON BEAM SIDE

i no.	Tension steel (top) No. and dia of bars	Tension steel (top) area in cm^2	Compn.steel (bottom) No. and dia in bars	Compn.steel (bottom) area in cm^2
	1-12 mm		1- 12mm	
	2- 6 mm	1.695	2- 6 mm	1.695
	2-10 mm		2- 6 mm	
	2- 6 mm	2.135	2- 10 mm	2.135
	2-12 mm		2-12 mm	
	2- 6 mm	2.825	2- 6 mm	2.825
	1-12 mm		1-12 mm	
	1-10 mm		1-10 mm	
	2- 6 mm	2.480	2-6 mm	2.480
	1-12 mm		1-12 mm	
	2- 6 mm	1.695	2- 6 mm	1.695
	2-10 mm		2-10 mm	
	2- 6 mm	2.135	2- 6 mm	2.135
	1-12 mm		1-12 mm	
	2- 6 mm	1.695	2-6 mm	1.695

Continued

TABLE 3.1 -(Contd)

DETAILS OF REINFORCEMENTS OF THE JOINTS

JOINT ON COLUMN SIDE

cimen	Tension steel (outside)		Compn steel (inside)	
	No. and dia of bars	area in cm^2	No. and dia of bars	Area in cm^2
	3 - 12 mm 1 - 6 mm	3.673	2-6 mm	0.565
	3- 12 mm 1 - 10 mm	4.175	2-6 mm	0.565
	4 - 12 mm	4.520	1-12 mm 2- 6 mm	1.695
	4 - 12 mm	4.520	2- 6 mm	0.565
	3- 12 mm 1 - 6 mm	3.673	2-6 mm	0.565
	3 - 12 mm 1 - 10 mm	4.175	2-6 mm	0.565
	3 - 12 mm 1 - 6 mm	3.673	2-6 mm	0.565

Materials

Granite metal of 1", 3/4" and 1/4" sizes in the ratio of 2:1:1, Kalpi sand and normal portland cement were used in the concrete mix, in the proportion of 1:1.1/2:3 by weight with a water cement ratio of 0.4. Mild steel bars of different sizes when tested gave an average ultimate strength of about 4300 kg/cm^2 and the cold drawn wires of 7mm diameterused as prestressed tendons had an ultimate strength of 16500 kg/cm^2. The typical stress strain graphs for 7 mm dia H.T.S. wire and 12 mm dia. mild steel bar are given in Figs. 3.4 and 3.5 respectively.

Casting and curing of frame specimens

The reinforcement and the prestressing tendons were assembled in a single wooden mould in order to maintain the uniformity for all the specimens.

Plywood partitions were inserted near the joints separating the joint portions from the beam and column. The concrete was placed in the moulds in small quantities and thoroughly vibrated with a needle vibrator. Simultaneously six numbers of 150 mm control cubes were also

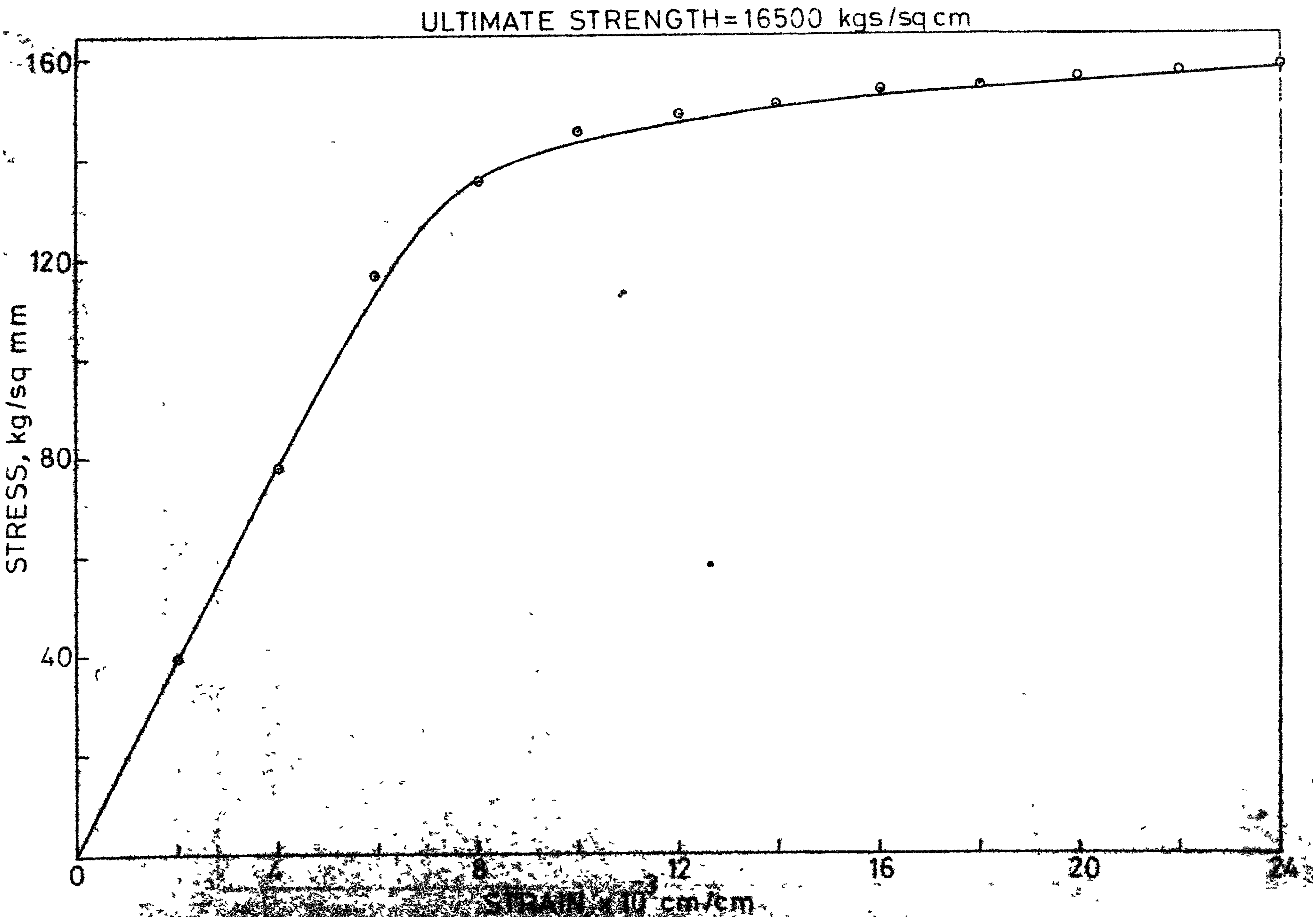


FIG. 3.4 STRESS-STRAIN CURVE FOR 7mm H.T.S. WIRE

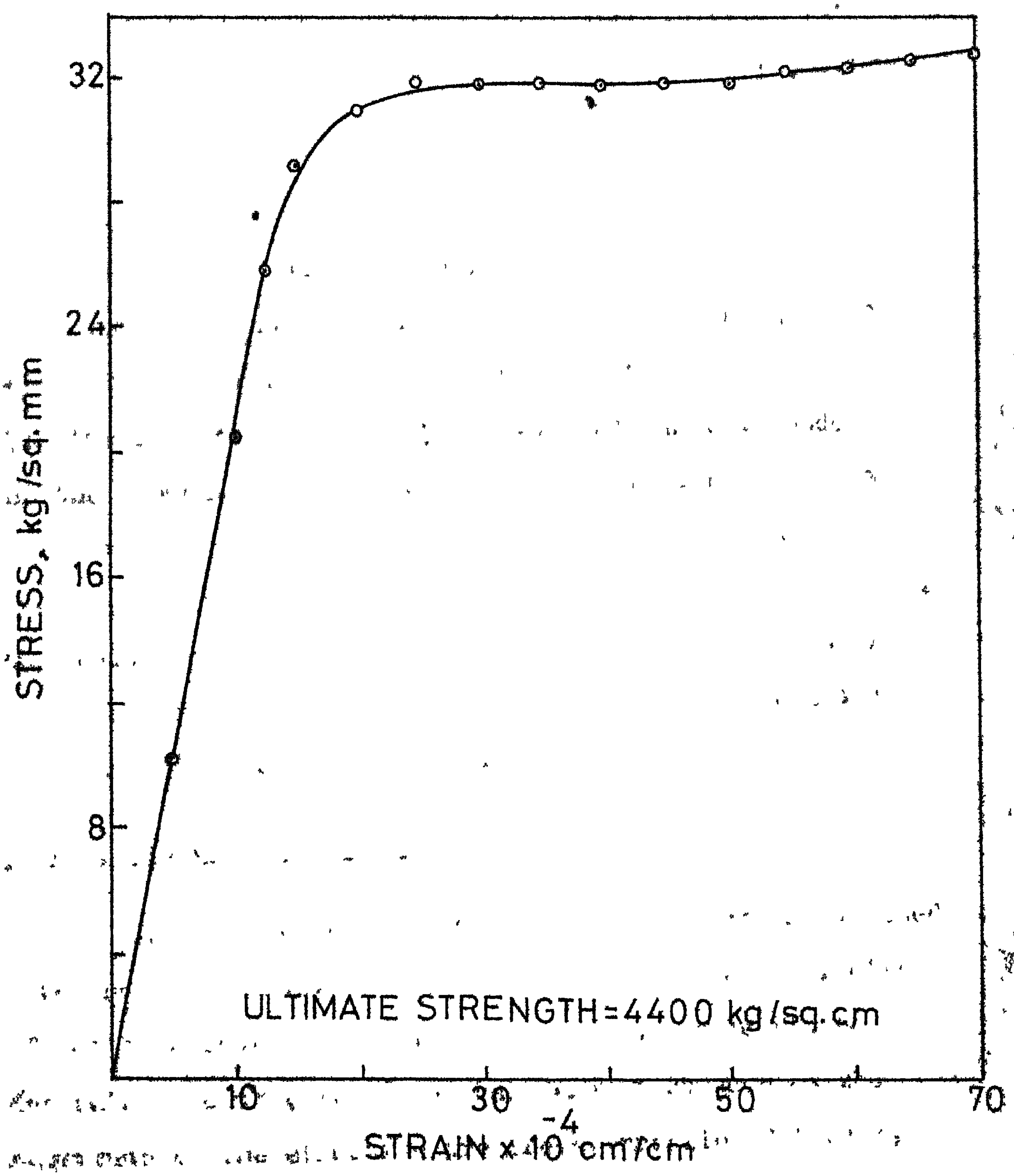


FIG. 3·5 STRESS-STRAIN CURVE FOR 12 mm M.S. ROD

cast along with the elements. After the setting time the elements as well as the control cubes were covered with wet gunny bags which were kept continuously wet by pouring water periodically.

After curing the elements in the moulds for two days, the plywood partitions were removed and the mild steel bars from the beam and column were welded together carefully. Then another batch of concrete mix was made with the same proportion of the materials as that made for the elements and the joints were cast along with six more control cubes. The curing of the specimen in the mould was continued for two more days under wet gunny bags. The mould was then separated and the specimen along with the 12 control cubes were cured under wet gunny bags for atleast 28 days before testing.

Casting and curing of beam specimens

The beam portion of the frame was to be prestressed with unbonded H.T.S. wires. The ultimate moment capacity of the beam section with unbonded tendons could not be determined accurately as several parameters such as the magnitude of the effective prestress, profile of tendon, shape of the bending moment diagram, length depth ratio

of member, amount of bonded non-prestressed supplementary mild steel etc., contribute to the variation in ultimate strength. In general some reduction factors were recommended to the bonded section to obtain the unbonded moment capacity. Some authors (45) have reported that unbonded post-tensioned beams with unprestressed mile steel reinforcement had strength equal to that of comparable bonded beams.

In view of all these factors it was proposed to cast the (simply supported) beam specimens with exactly the same arrangement of reinforcement as that in the beam portion of the frame except the additional joint reinforcements on either end of the beam, and test it to destruction to find the actual ultimate moment capacity. The detailed calculations for the same are given in Appendix-B. The beam specimens were cast and cured in the same manner as the frame.

Prestressing

One day before testing specimens were allowed to dry. They were then cleaned with a brush and white washed to facilitate the easy observation and marking of cracks.

The beam portion was then prestressed using Gifford-Udall C.C.L. system and the tendons were anchored at ends using 1/4" mild steel plates. The average prestressing force applied as indicated on the jack dial was 4600 kg. for each wire.

3.2 Loading cycle :

The live loads on a structure consist of pedestrian and vehicular traffic on bridges, weight due to inhabitants, furniture and other equipment on buildings, snow loads, wind loads, earthquake loads etc. They are of highly variable nature and may vary in their position, magnitude direction and combinations. While the normal live loads will be acting many times, probably several lakhs of times during the life span of the structure, the combined effect of different live loads will be felt on the structure rarely.

Since it will be very uneconomical to design a structure for this combined load effect which will be occuring only rarely, the design load will be in general selected at a level upto which the actual loads will be reaching frequently. But at the same time the effect of the combined load cannot be ignored. The loading schedule on the specimens was adopted keeping the above factors in view.

The frame specimen was subjected to vertical loads P each at one third points on the beam and a horizontal load equal to a fraction of "P". Vertical loads represent the gravity loads inclusive of dead load and normal live loads. Horizontal load represents the normal wind load. It is difficult to estimate the ratio of horizontal load to vertical load since it depends on many factors. A horizontal load of P/12 for some specimens and P/6 for other specimens was chosen arbitrarily.

The vertical as well as horizontal loads were applied through hydraulic jacks connected to the pulsator. For the purpose of testing it was assumed that the design load on the structure will occur about million times in the span of its life and that the combined load which was assumed to be 33.1/3% more than the working load will occur once in about 150 cycles. Since it was difficult to raise the level once in every 150 cycles, the combined loads were given about 600 times in every 0.1 million cycles. At the frequency of about 600 cycles per minute the peak load pulsations were given for one minute approximately in every three hours of running of the pulsator. The lower limit of pulsations both at the working

load level and the combined load level was taken as the dead load level on the structure.

The fixed load (dead load and fixed superimposed loads) P_d, was assumed to be about 0.5 times the normal working load P_w. The combined load P_p, termed as peak load was assumed to be about 1.1/3 times the working load. Details of loads applied on the test specimens are given in Table 3.2. The typical loading cycle on the specimen in a day was, as indicated in Figure 3.6. The pulsations were given about 6 hours a day and for the rest of the period in the day the specimen was left to undergo relaxation.

Re evaluation of collapse load based on the tested cube strengths :

The load values P_w, P_d and P_p on the specimen were obtained using the ultimate moment capacities re-evaluated based on the actual cube strengths. Detailed calculation of this reevaluation for specimen No.1 are given in Appendix-C. The ultimate moment capacities of the sections for all the frames are given in Table 3.3.

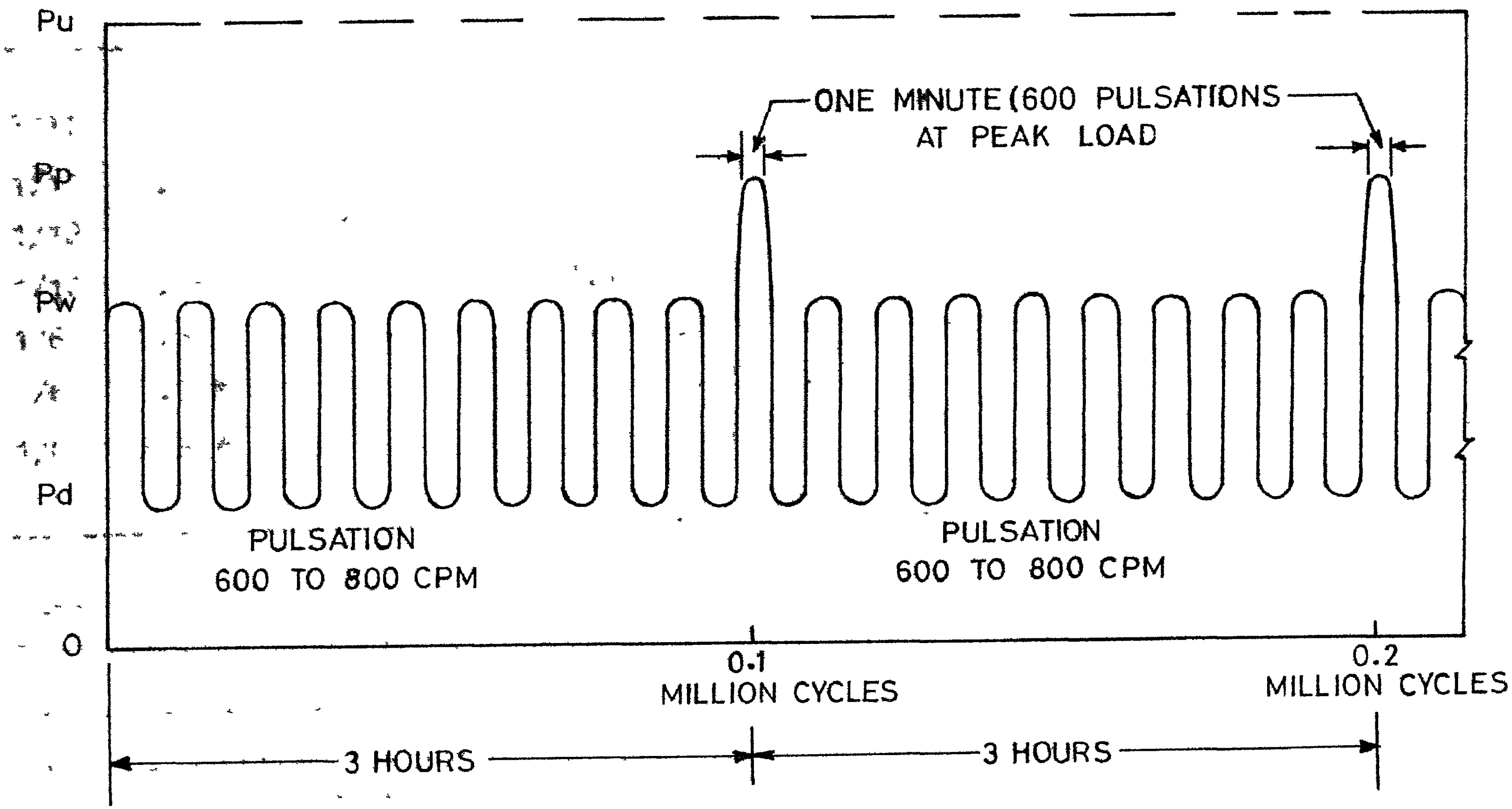


FIG. 3.6 LOADING SCHEME PER DAY

TABLE 3.2 DETAILS OF LOADS ON THE TEST SPECIMENS

Speci-men No.	K	P_u^T (Tonnes)	P_u^E (Tonnes)	$\frac{P_u^E}{P_u^T}$	Load Applied in Tonnes P_d	P_w	P_p	Loads as Per-cent of P_u^E P_d	P_w	P_p	$\frac{P_p}{P_w}$	$F_o = \frac{P_u^E}{P_w}$	No. of pulsations in millions
1	1/13	4.53	5.45	1.200	1.09	2.46	3.28	20.0	45.3	60.2	1.333	2.20	1.0
2	1/12	4.33	4.75	1.100	1.09	2.29	3.06	22.9	48.2	64.5	1.333	2.07	1.0
3	1/12	5.12	4.91+	0.953	1.09	2.53	3.38	22.2	51.5	68.7	1.333	1.94	1.0
4	1/12	4.16	4.47	1.073	1.18	2.95	3.93	26.4	66.0	88.0	1.333	1.51	0.3
5	1/6	3.85	5.35	1.390	1.22	3.14	4.03	22.8	59.8	75.3	1.290	1.70	0.1
6	1/6	3.95*			1.18	2.95	3.93				1.330		0.2
7	1/6	3.85*			1.15	2.86	3.82				1.330		0.3

K = Ratio of Horizontal load to single vertical load; P_u^T = Ult. Load Theoretical;
P_u^E= Ultimate Load exptl.; P_d = Dead load; P_w = Working load; P_p = Peak load;

F_o= Overall load factor.

+ Specimen - 3 failed (secondary failure) in shear due to opening of welds in shear connectors
* Specimens 6 and 7 failed (secondary failure) due to opening of welds at the beam-column joint.

TABLE 3.3

ULTIMATE MOMENT CAPACITIES OF THE SECTIONS

Specimen No.	Cube strength of elements Kg/cm^2	Mu of beam at the critical section in ton.m.	Cube strenth of Joint in Kg/cm^2	Mu of joint on beam side ton. m.	Mu of joint on column side ton. m.
1	540	1.179	517	1.499	1.440
2	400	1.097	400	1.550	1.430
3	540	1.179	500	1.805	2.009
4	440	1.134	335	1.586	1.290
5	503	1.165	507	1.494	1.400
6	425	1.100	420	1.500	1.400
7	503	1.165	507	1.494	1.400

3.5 Test set up

The general arrangement of testing is shown in Figs. 3.7 to 3.9. The horizontal set up was adopted. Two brackets were fixed to the test bed at 150 cms apart forming supports to the frame. The hinge action at each support was provided by means of a pin and the v-blocks. The pins were welded to the brackets so that the distance between their centres is exactly 150 cms. when the specimen was kept in its proper position, the pin welded to the bracket fits into the groove of the V-blocks attached to the base of the columns. The arrangement prevents lateral movement and allows only rotation about the axis of the pin. The typical hinged joint is shown in Figure 3.10.

As the lateral load was only a fraction of vertical load a jack of a smaller capacity was required. A suitable lever arrangement was provided as shown in Figure 3.7, so as to get the exact required load at the point of application, since the smallest capacity available was of 16000 lbs. All the three jacks were connected to 'Riehle Sc 10 Pulsator'. Stiffening tie bars were provided in the supporting brackets to minimise the support deformations.

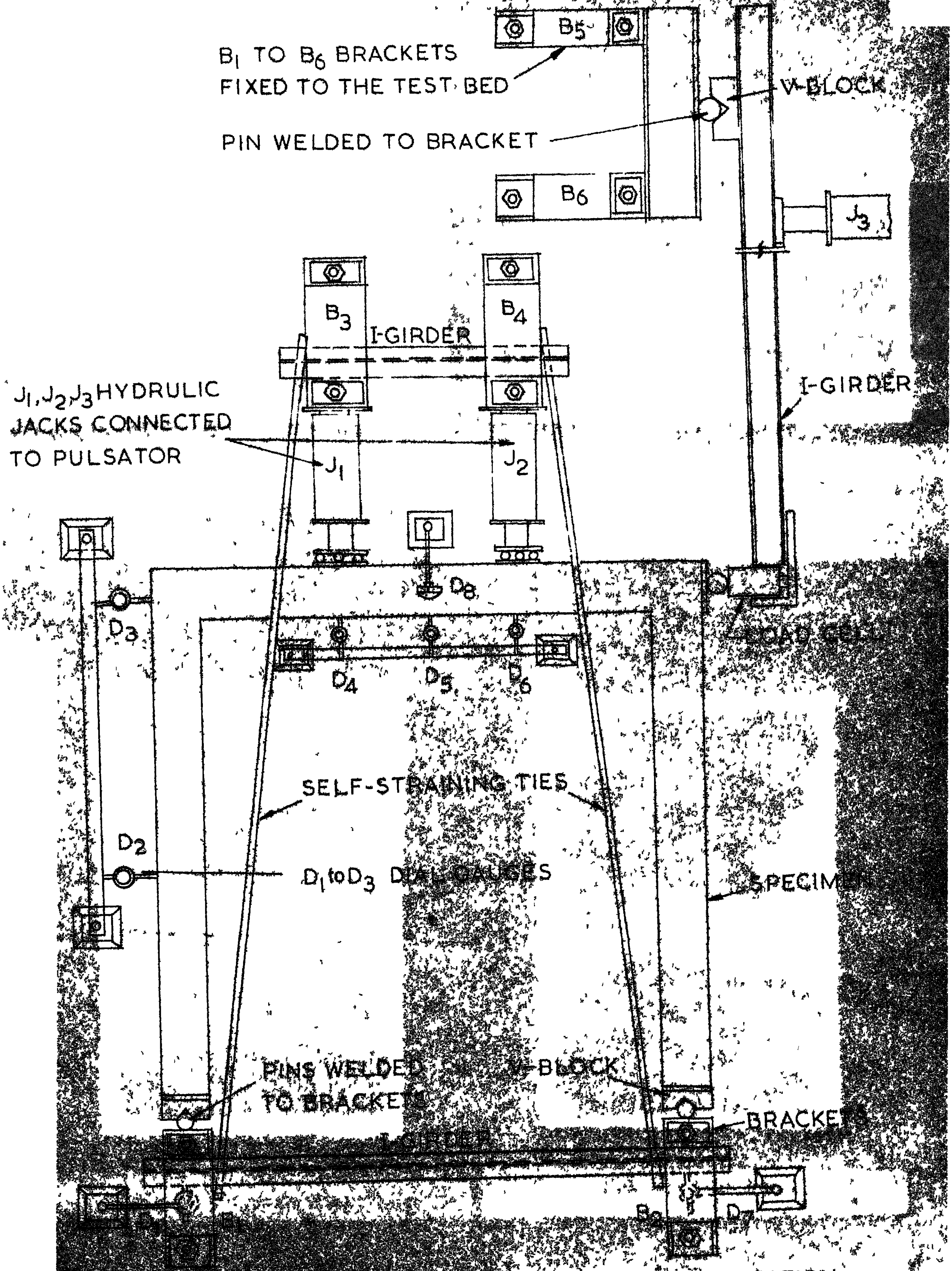
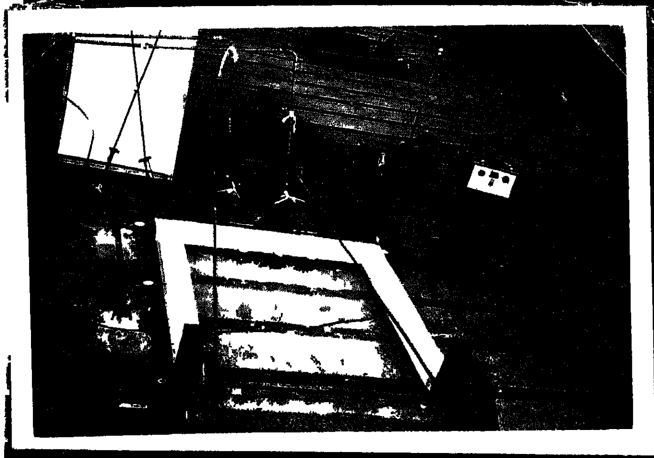


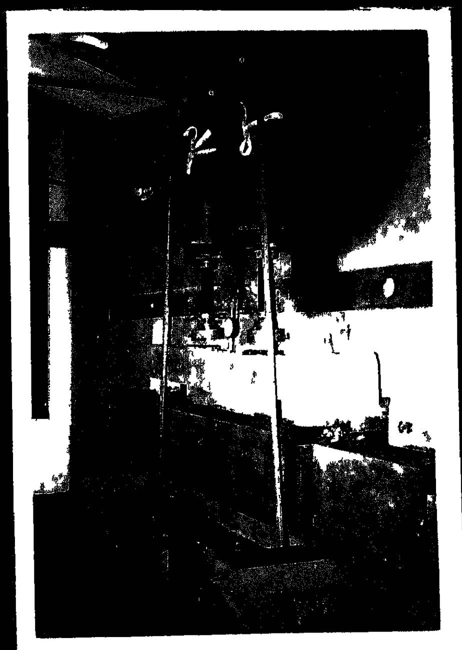
FIG. 3.1 TEST SET-UP AND INSTUMENTATION
(HORIZONTAL SET-UP)



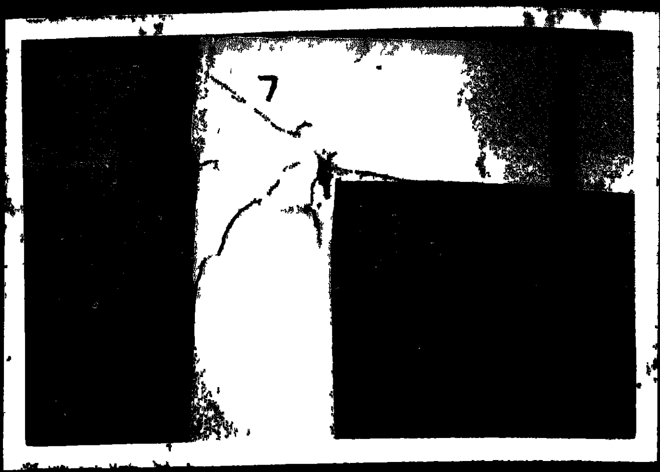
General view of test set up

3.10 Typical arrangement of the hinged support of the frame

3.9 Another view of test set up



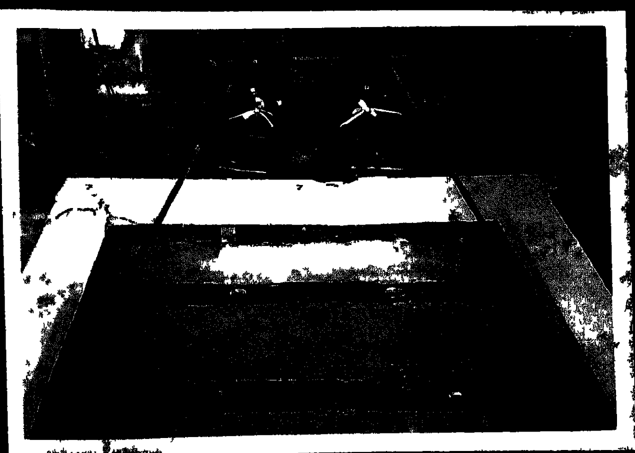
3.11 Set up for beam testing



5.12 Typical crack pattern
(Specimen 7)
at the joint

5.13 Crack pattern at the
joint of specimen 5

5.14 Typical crack pattern
(Specimen 5)
in the beam portion



5.15 Typical failure mechanism

3.6 Instrumentation :

The instrumentation adopted for the testing is shown in Figs. 3.7 and 3.8. A set of dial gauges were provided to measure the deflections at various points in the specimen and a load cell at the right hand top corner to measure the horizontal load. Gauges D_2 and D_3 were provided to measure the horizontal deflections at top and middle of the left column. Gauges D_4 to D_6 were provided to measure the vertical deflections at one third points and centre of the beam. Gauges D_1 and D_7 were meant to check the rigidity of the supports and sinking or settlement if any at the hinge supports. Dial D_8 was provided to note the out of plane movements of the frame. The test cylinders were calibrated with the help of proving rings and found to check very closely.

3.7 Testing :

<u>Testing programme</u> :

Two beam specimens were tested to ascertain the accuracy of the moment capacities of the prestressed concrete unbonded section used for the beam portion of the frame specimen. Altogether seven frame specimens, designed with different joint load factors, were tested.

Horizontal load chosen was P/13 for the first specimen, P/12 for the next three specimens and P/6 for the last three specimens. The specimens were subjected to the loading cycle mentioned earlier. They were first subjected to the static loads upto the peak load level twice or thrice and then given pulsations varying from dead load level to working load level with peak load pulsations 600 times in every 0.1 million cycles. A static load test was conducted upto collapse of the specimen after the required number of pulsations were given.

Static test on beams :

The set up for beam testing is shown in Figure 3.11. Beam specimens were tested under static load to collapse while observing the deflections at centre for various levels of the load. The load deflection curves for the two beam specimens are shown in Figures 3.16 and 3.17.

Testing of the frame specimens :

After white washing and prestressing, the specimen was moved to the test bed, positioned and levelled properly. Positions where the deflection has to be measured were marked on the specimen in pencil and the dial gauges were set up. The load cell was fitted at the right hand top

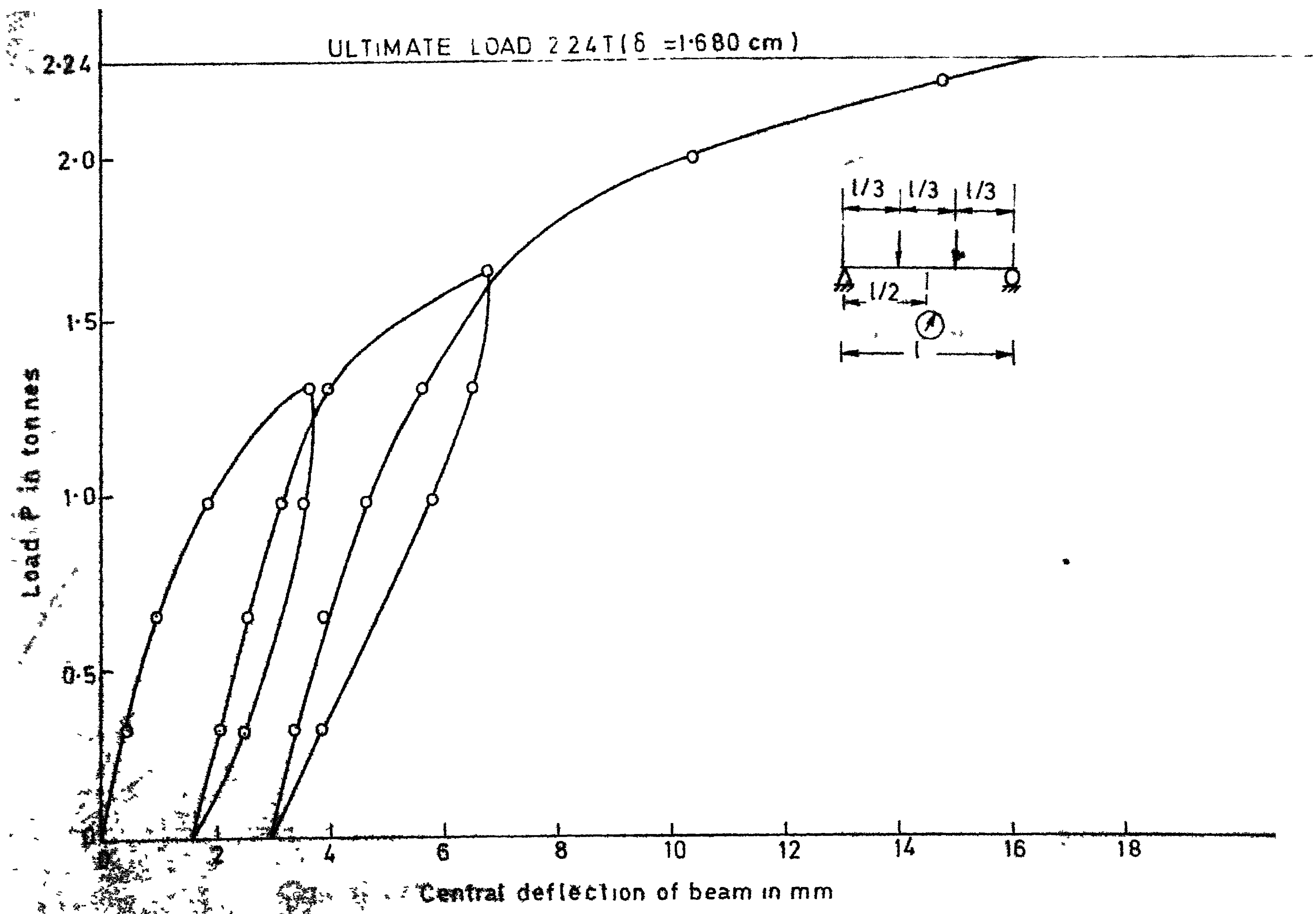


FIG. 3.1[illegible] LOAD DEFLECTION CURVE OF UNBONDED (PRESTRESSED) BEAM-1

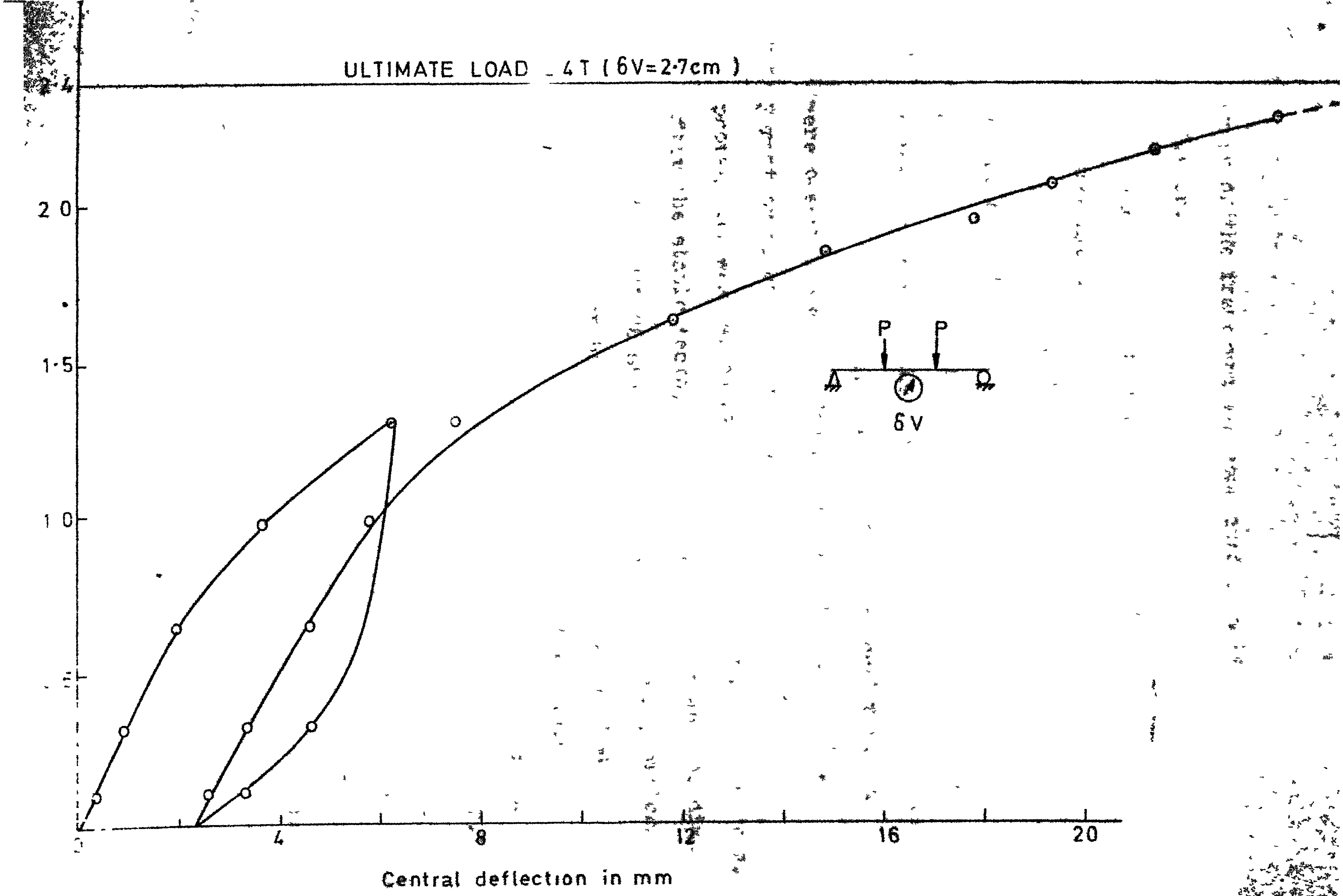


FIG. 3·17 LOAD DEFLECTION CURVE OF BEAM-2 (WITH BONDED H.T.S. WIRE WITHOUT PRESTRESSING FORCE)

corner of the frame and was connected to a strain indicator.

The load was then applied simultaneously through the three hydraulic jacks by operating the pulsator for static condition. The load was increased gradually in increments of 5% of the jack capacity upto a load level slightly lower than the peak load level, and unloaded. In the second cycle the load was increased upto peak load and reduced. The dial gauge and the load cell readings were observed and recorded for different load levels. Formation of cracks during load increases and their propagation was marked with a black pencil on the specimens. After the static tests, load cell was removed and the dial gauges were disengaged without disturbing their positions by putting rubber bands around them. Then the dynamic loads were applied and the frequency of the pulsations was gradually increased until it reached a level of 600 to 800 cycles per minute. After application of about 0.1 million cycles of pulsation in about 3 hours, the upper load limit was gradually increased in about 2 minutes upto a predetermined peak load level, kept constant for about one minute to give 600 cycles and then reduced gradually in about 2 minutes to the working load level.

Like this peak pulsations were given once in every 0.1 million cycles. Deflection measurements were made everyday before and after the application of pulsating loads, to observe the creep relaxation that the specimen had undergone overnight.

After the completion of the desired number of pulsations, the specimen was tested to static collapse. Crack propagation was duly marked on the specimens. The dial gauges were removed just before collapse to avoid damage. Similar tests were conducted on the remaining specimens.

The loading levels in the first three specimens were approximately same. P_w was about 45 to 52% of P_u, and P_p was 33.1/3% excess of P_w. All the three specimens have survived million cycles and 10 spells of peak loads without any damage. During the final static load cycle, the specimen-3 had suffered a failure in shear near about the theoretical ultimate load value. On inspection it was found that failure was due to the failure of welds of the shear connectors. Specimen-4, with the same horizontal load P/12, was tested at higher load levels of P_w = 66% of P_u and peak load 33.1/3% more than P_w. After 0.3

million cycles of normal pulsations and 3 spells of peak pulsations, the specimen was found to develop wide cracks at peak load level which were not closing completely on removal of the load. The pulsations were then stopped and a static test was conducted upto collapse.

The tests on last three specimens were conducted with horizontal load P/6 and with increased levels of P_w of about 60 to 75% of P_u and $P_p = 1.33\ P_w$. Specimen 5 survived only 0.1 million cycles before it developed wide cracks. Specimens 6 and 7 suffered a secondary failure by the failure of the left hand side joint between beam and column, which was found to be the result of opening of welds.

3.8 Results and Discussions :

Figures 3.16 and 3.17 illustrate the load deflection behaviour of the beams. Figures 3.18 to 3.24 illustrate the load versus vertical deflection behaviour of specimens 1 to 7. A typical, load versus horizontal deflection for the specimen No.4 is shown in Fig. 3.25.

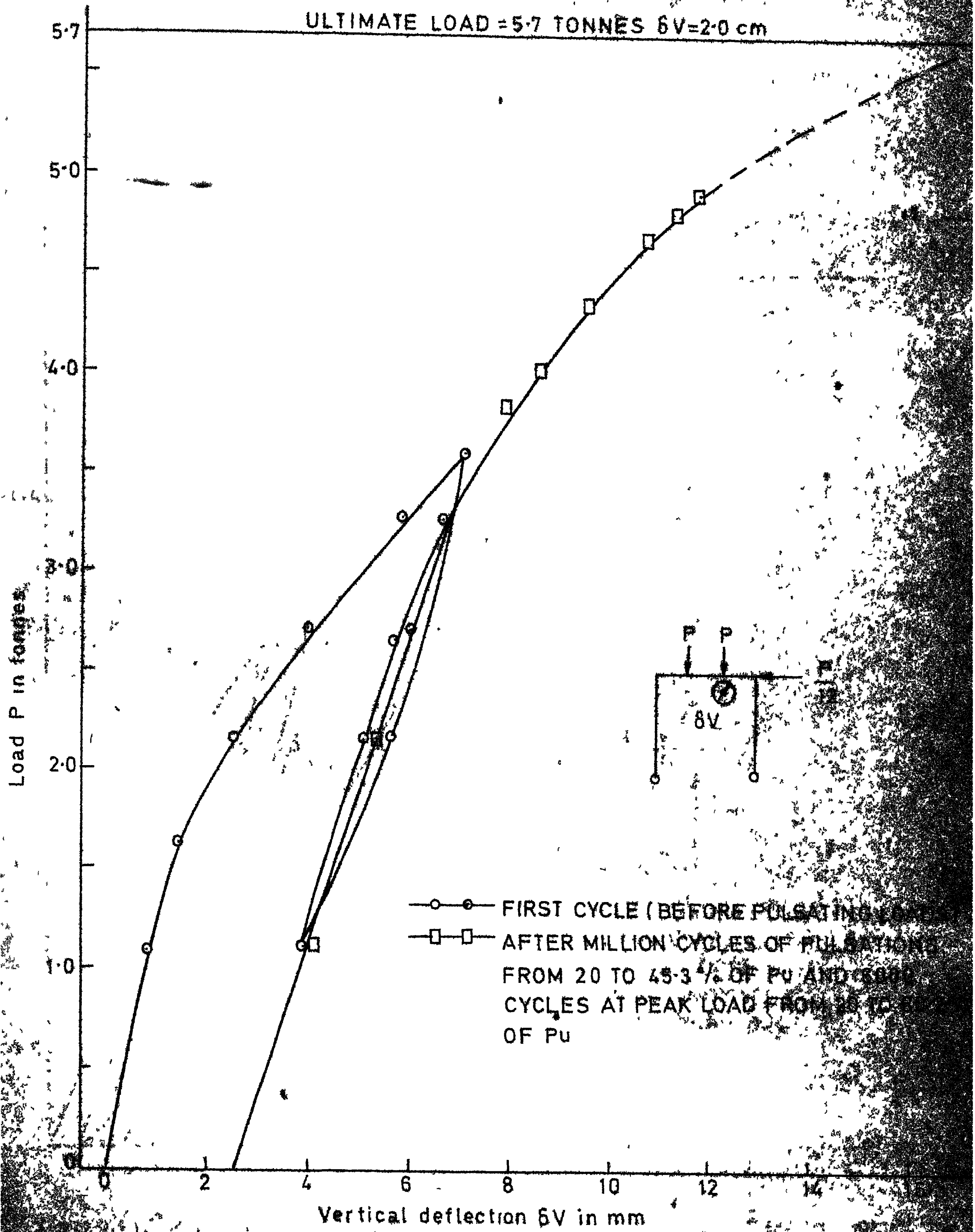


FIG. 3·18 LOAD DEFLECTION CURVE (P Vs δV) FOR SPECIMEN 1

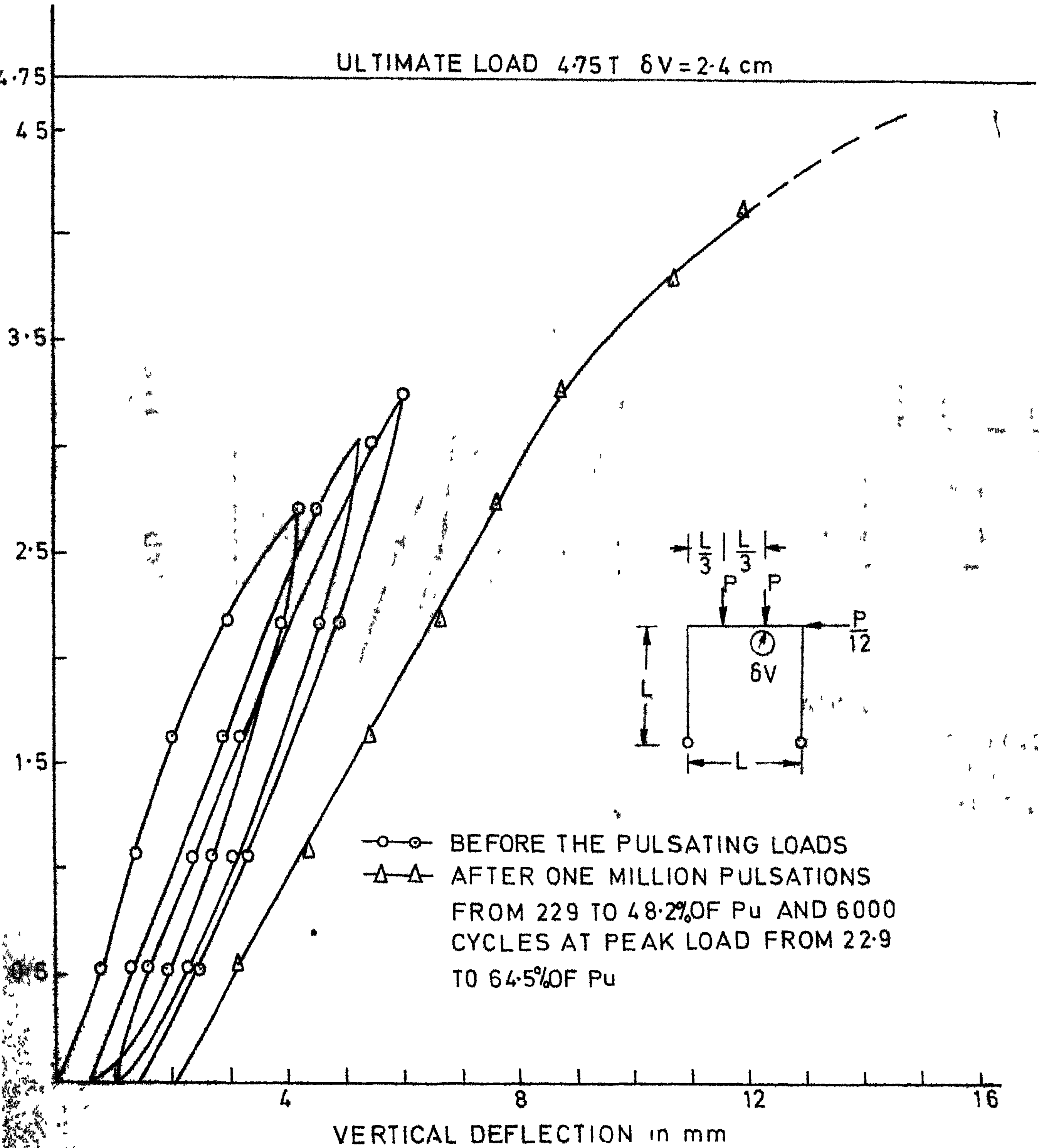


FIG. 3·19 LOAD DEFLECTION CURVE (P Vs δV) FOR SPECIMEN-2

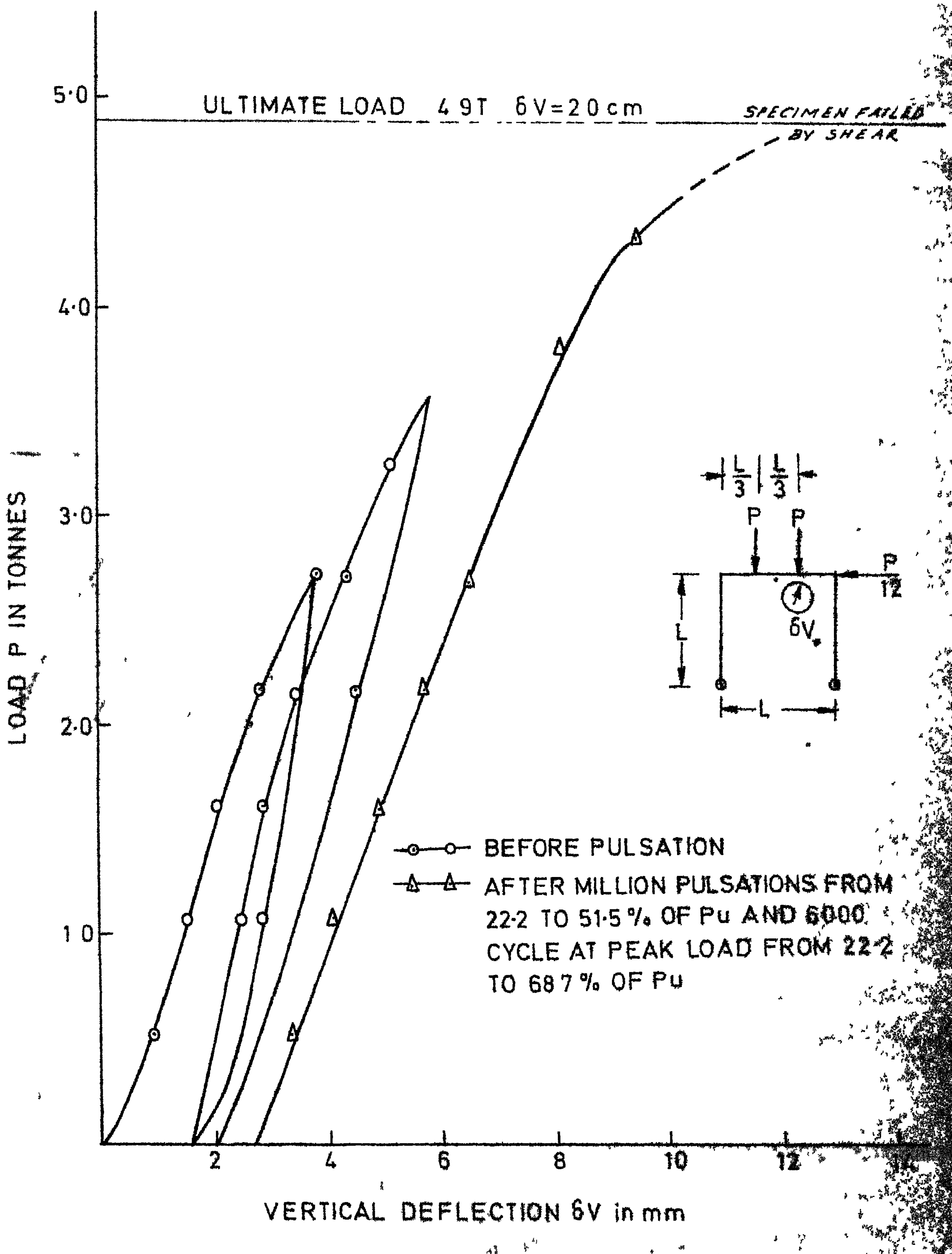
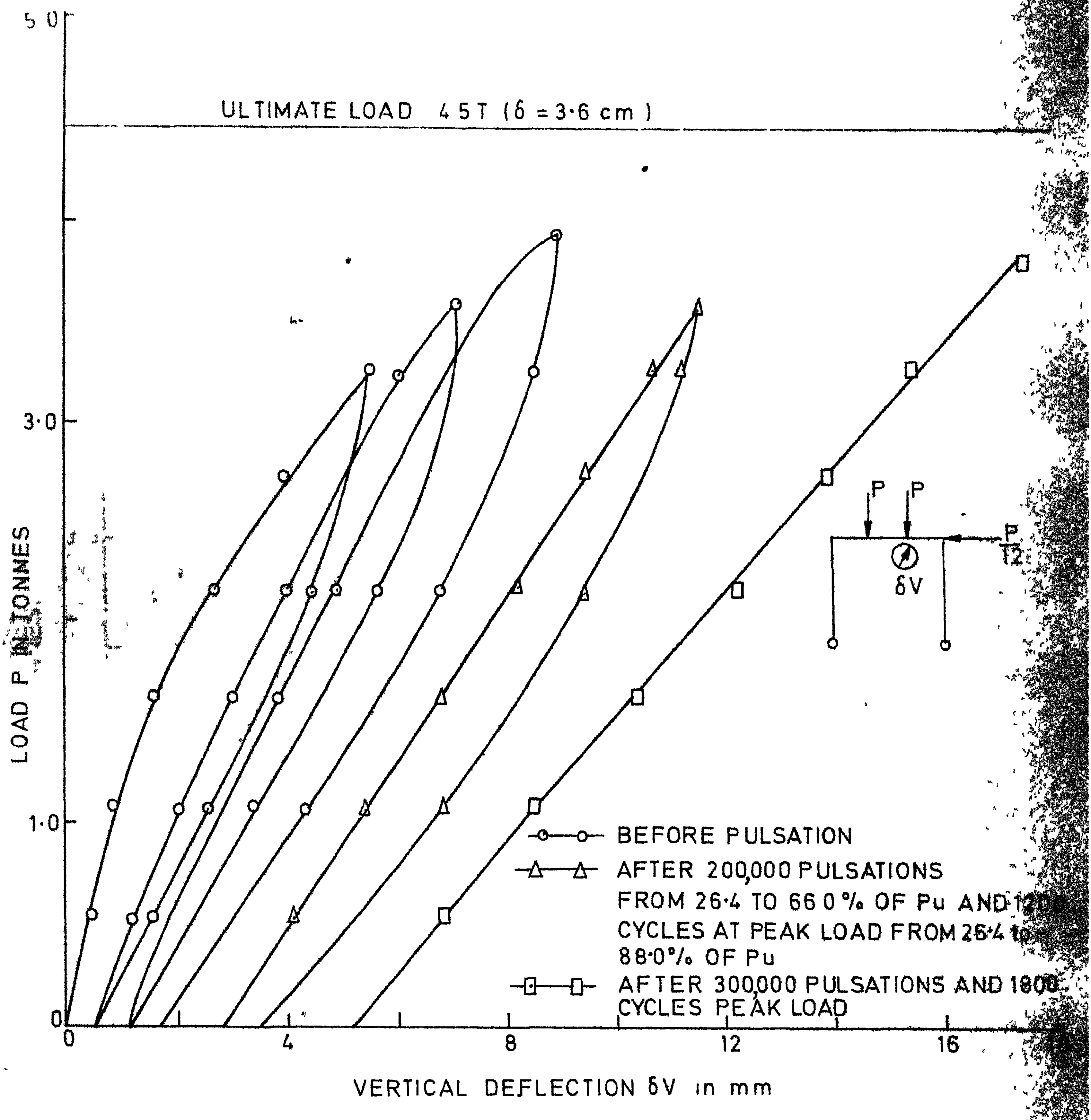


FIG. 3·20 LOAD DEFLECTION (P Vs δV) CURVE OF SPECIMEN-3



FIG· 3·21 LOAD DEFLECTION (P Vs δV) CURVE OF SPECIMEN 4

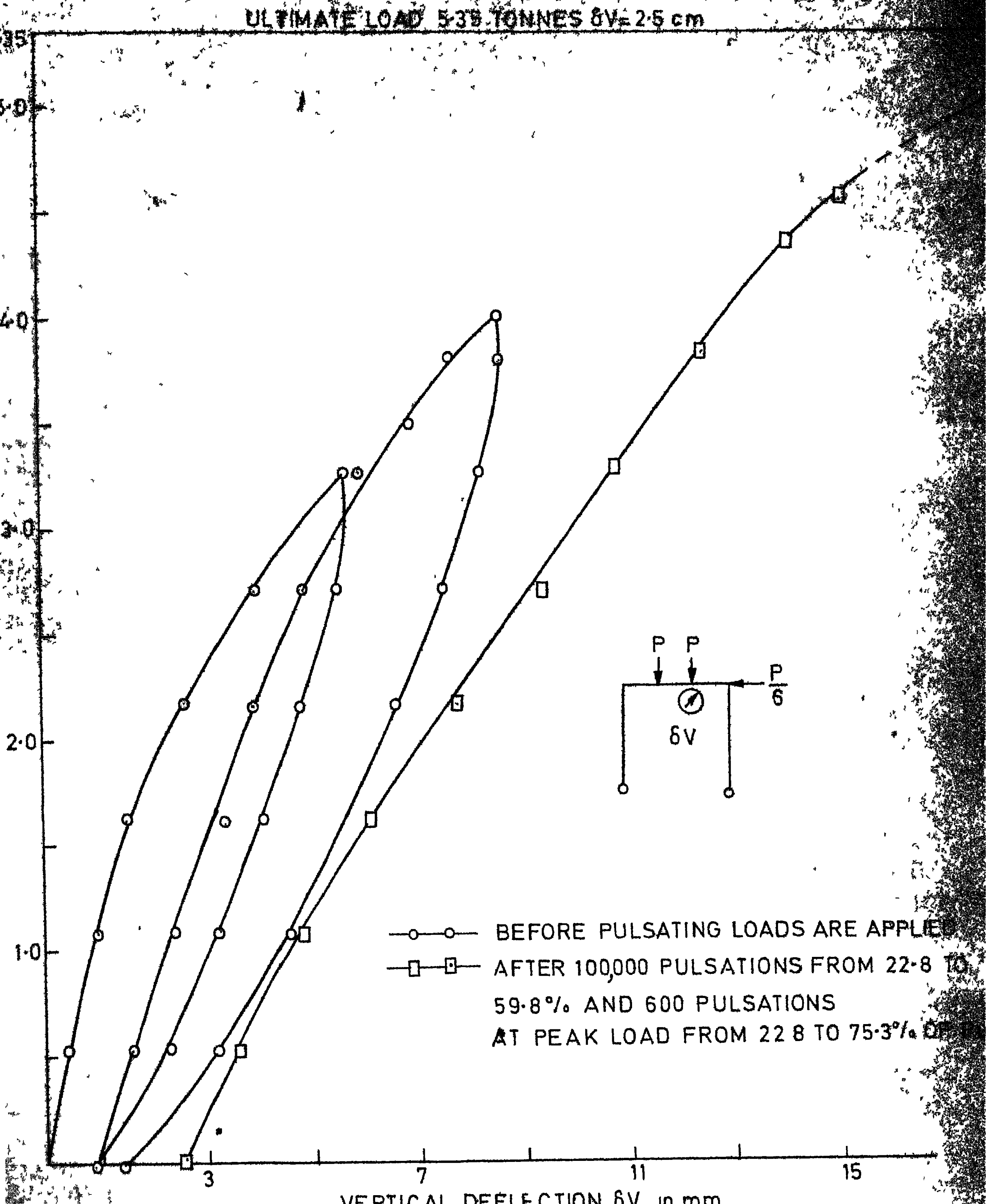


FIG. 3·22 LOAD DEFLECTION CURVE (P Vs δV) OF SPECIMEN-5

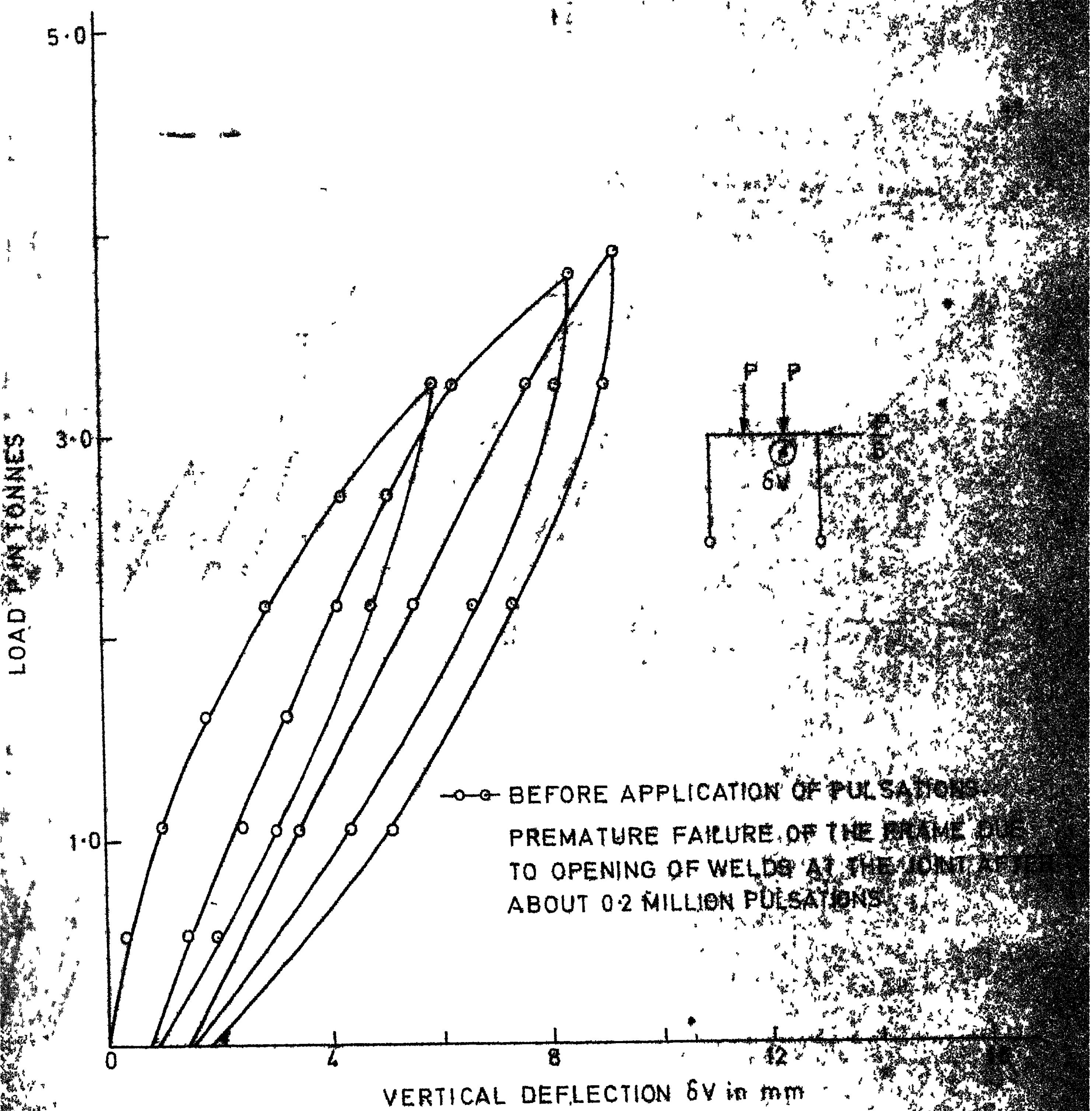


FIG. 3·23 LOAD DEFLECTION CURVE (P Vs δV) FOR SPECIMEN-6

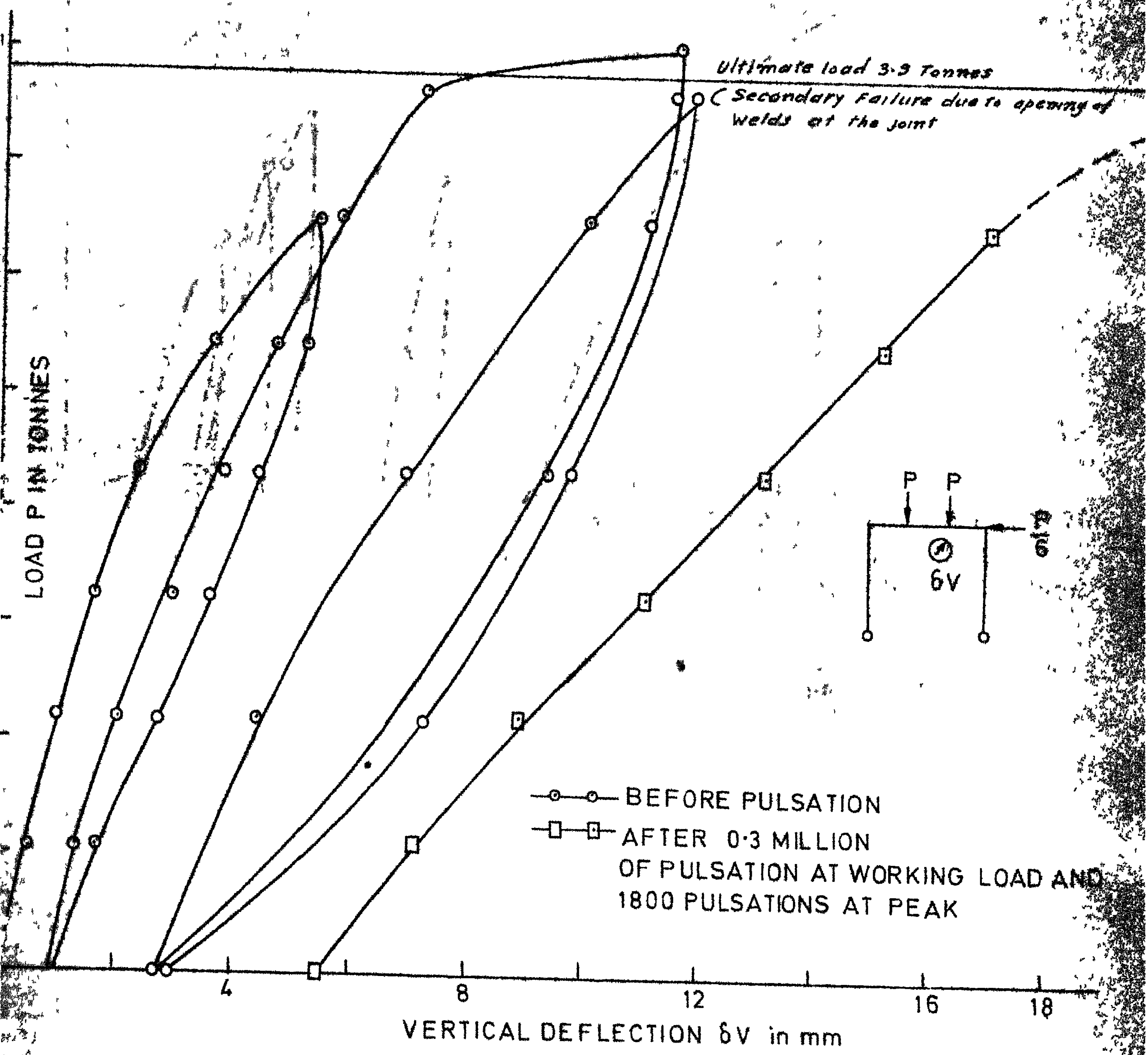


FIG. 3·24 LOAD DEFLECTION CURVE (P Vs δV) OF SPECIMEN-7

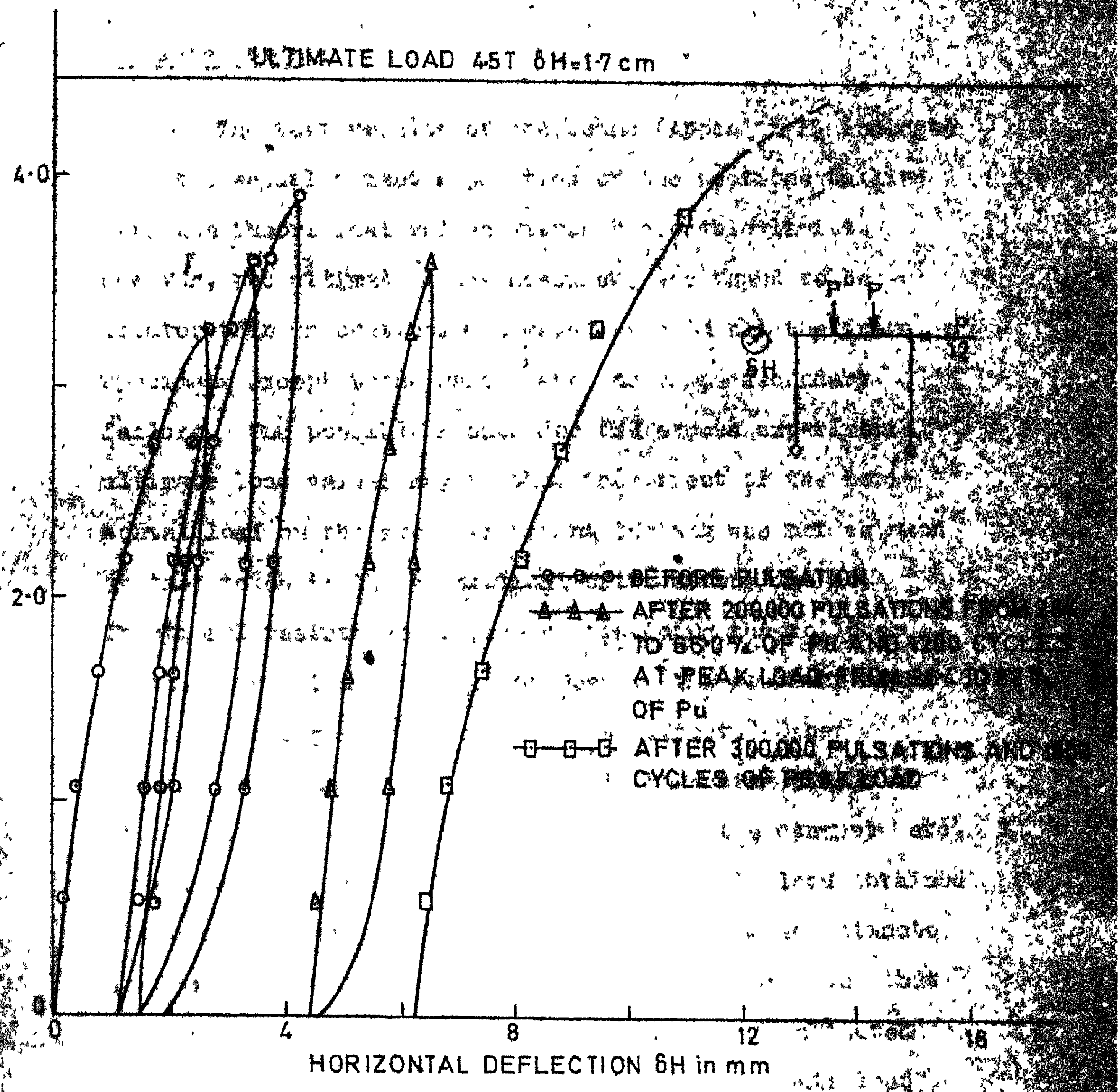


FIG. 3·25 LOAD DEFLECTION (P Vs δ H) CURVE OF SPECIMEN 4

Ultimate loads :

The test results of the beams (Appendix B) indicate that the actual moment capacities of the sections tallied with the theoritical values within reasonable limits. However, the ultimate loads measured, are found to be greater than theoretical ultimate loads in all the frame specimens except those which have undergone secondary failure. One possible reason for the excess experimental ultimate load values may be that the effect of the horizontal load on the specimen during testing was not as much as that taken in the theoretical calculations due to frictional resistance inspite of providing the rollers at the point of application of loads. The specimen might not have undergone the complete sway effect. Other possible reasons might be the strain hardening effect of steel, uncertainties of the uniformity of the concrete etc. However, since in all the cases, the ultimate load obtained after the pulsations was more than the estimated ultimate load under the static conditions it can be concluded that the ultimate strength of the specimens was not affected by the pulsating loads. The experimental ultimate load was taken as the basis for the calculation of load factor and other particulars given in Tables 3.1 and 3.2.

Failure mechanism :

Failure by the combined mechanism was observed in all the specimens except in the secondary failure cases. The plastic hinges were formed at the critical points, one at the point of action of right side vertical load in the beam and the other at the left side joint. The typical failure mechanism was as shown in Fig. 3.15.

Crack Pattern

Crack pattern in all the specimens of the mechanism failure, was found to be almost similar. The first crack generally formed at the right side vertical load. Subsequent cracks appeared in between the two vertical loads in the beam. Cracks in the joint appeared at a later stage. There were practically no cracks in the main column portion except near the joints. All the cracks were found to widen under the pulsations and were found to close on removal of the load. The cracks in the joints have developed in the diagonal directions. The typical crack pattern in the beam and joint portions were as shown in Figs. 3.12 to 3.14.

Cumulative damage :

Specimens 4 and 5 have been subjected to cumulative damage. The damage was in terms of the progressive widening of cracks much beyond the permissible crack width as the pulsation was continued. The failure of the specimen might have occured if the pulsations were continued. This was due to the higher levels of applied pulsating loads. The working loads were 60 to 75% of P_u, the peak loads being 33-1/3% more than P_w. At lower load levels of working load 50% and peak load 65% of P_u, as were adopted in specimens 1 to 3, no effect of cumulative damage was felt.

However, cumulative deflections were observed in all the specimens, with the increasing number of pulsations. The rate of increase in the cumulative deflection was more in the earlier stages of pulsations and decreased with increasing number of pulsations. Further from the readings of the deflections before and after the pulsations every day it has been observed that there is an amount of recovery in these deflections overnight when the specimens were left without load. This may be termed as creep recovery. This creep recovery was also found to be greater in earlier stages than in the later stages of pulsations.

Stiffness of the specimens :

From the Figures 3.18 to 3.25 it is observed that there is a flattening of the gradient of the load deflection curves after pulsating loads, indicating the reduction in the stiffness of the specimen. The loss of stiffness increased with the number of cycles and is comparatively less in the specimens with higher joint load factors. However the reduction in stiffness is not as much as to affect the serviceability of the structure

Load factors :

Load factors for the specimens obtained from the moment capacities calculated based on the actual cube strengths are given in Table 3.4. Since the experimental ultimate loads on the specimens was found to be higher than the theoretical values the load factors were also calculated based on the experimental ~~to~~ ultimate load and are listed in Table 3.5. Details of calculation are given in Appendix-C. As discussed earlier, at higher load levels i.e., at lower load factors as in specimens 4 & 5 the structure had lost its serviceability by developing wide cracks beyond the permissible limits though the ultimate strengths do not seem to have been affected.

TABLE 3.4(a) DETAILS OF LOAD FACTORS BASED ON THEORETICAL ULTIMATE LOAD OF THE SPECIMENS

Specimen No.	Beam Load Factor F_b	Column Load Factor F_c	Joint Load Factor F_j	Overall Load Factor F_o
1	1.5	1.7	2.27	1.84
2	1.5	1.7	2.39	1.90
3	1.5	1.7	2.72	2.02
4	1.2	1.7	1.67	1.41
5	1.1	1.7	1.37	1.23
6	1.1	1.7	1.46	1.34
7	1.2	1.7	1.50	1.35

TABLE 3.4 (b) DETAILS OF LOAD FACTORS BASED ON EXPERIMENTAL ULTIMATE LOAD OF THE SPECIMENS

Specimen No.	F_b	F_j	F_o
1	1.8	2.72	2.21
2	1.65	2.63	2.07
3	1.44	2.60	1.94
4	1.29	1.79	1.51
5	1.53	1.91	1.70

NOTE : Load Factors for specimen 6 & 7 are not given since P_u^E is not available owing to their premature failure.

CHAPTER IV

CONCLUSIONS

4.1 Load factor design :

There is a need to introduce in concrete structures, the concept of adopting different load factors to different components, depending on the extent of uncertainties in their design assumptions, difficulties in their construction, possibilities of frequent stress concentrations on a particular component etc. This concept which is known as 'load factor' design is being adopted in the design of steel structures. It provides a more rational approach to the provision of safety allowances in the design for various components rather than a single factor for the whole structure which may sometimes lead to an undersafety design to a particular component in the structure while the other components are oversafe. Beams and columns are the principal elements of a framed structure. The strain energy per unit volume in a member under a primarily compressive stress is much more than the strain energy per unit volume of member under flexure. This suggests a need

for a higher load factor in a column compared to that of beam.

The need for a load factor design is felt much more in precast concrete construction. In most of the precast concrete construction, the beam, column and other elements are cast in a factory under quality control while the joints are cast in situ, most of the times at very inconvenient positions and under difficult circumstances, making it impossible to have the same amount of quality control in the construction of the joint as that of the elements. In view of all these factors provision of an increased load factor for a joint as compared to the load factor of the elements is very essential. ACI code recommends a load factor for the joint not less than 10% more than the load factor of the joining elements in precast construction.

Most of the codes have yet to adopt this concept and provide in their design specifications either for reinforced concrete structures cast-in-situ or precast. In working stress design this concept was indirectly introduced in almost all the codes when a lower allowable stress was adopted for the concrete under direct compression than that in bending. But in the inelastic methods

of design, most of the codes have yet to provide for this concept. To some extent ACI code has made provision for the load factor design in the ultimate strength method (Article 1504 in ref. 2), by adopting different capacity reduction factors, to different elements. This factor shall be 0.90 for flexure, 0.85 for diagonal tension bond and anchorage 0.75 for spirally reinforced columns and 0.70 for tied compn. members.

4.2 Conclusions from experimental investigations :

Choice of load factors for components of the Structure :

The actual strength of cubes has differed from the initial assumed value of 450 kg/cm^2 in the design of specimens. Therefore the actual load factors have changed from those used in the design calculations. Thus the investigation could not be done in the range of desired load factors. From the limited experimental investigation carried out it was found that a beam load factor as low as 1.4 was found to be satisfactory provided the overall load factor is more than about 1.7. Therefore a beam load factor of 1.4 to 1.5 is recommended for adoption in

the design. The loading pattern in the experiment was such that there is no column failure and so the effect of column load factor could not be studied. A column load factor of 1.7 is quite safe as there was no column failure with this load factor. The joint load factor of about 30 to 40% more than the beam load factor appears to be more than satisfactory. While selecting the relative values of load factors for the elements, the overall load factor should always be kept in view.

Cyclic loading and serviceability

The behaviour of a reinforced concrete structure designed with an overall load factor of 1.8 to 2.0 for the working loads was satisfactory with reference to the serviceability as well as the strength, when acted upon by cyclic loading, varying from dead load level to working load level about million times, and occasional peak loads about 33% more than the working load, representing the combined action of some live loads. Therefore the overall load factor to be adopted for the combined loads can be obtained by dividing the above recommended load factors for the working load by 1.33. Thus an overall load factor of about 1.3 to 1.5 appears to be satisfactory for

the combined loads. In a similar manner the load factors under combined load conditions can be deduced as of 1.1 to 1.2 for beams, 1.3 to 1.5 for joints and 1.2 to 1.4 for columns.

The ultimate strength of the specimen is not affected even when it is subjected to pulsations at peak load as high as 90% of its ultimate load. But from load deflection curves it is observed that there is reduction in the stiffness of the specimen with the increasing number of pulsations. The reduction in the stiffness of the specimen is comparatively less for higher joint load factors.

Secondary failure in the joints :

In case of two specimens which were subjected to higher load levels of working load being 60 to 66% of the ultimate load and peak load being 75 to 88 percent of ultimate load, there was joint failure due to opening of welds. This failure of welding, may be the result of the improper welding due to the inconvenient position, when the two elements were joined or it may be due to partial fatigue due to stress concentrations. Either of these cases is possible in actual construction of precast concrete structures , thus justifying the need for the extra provision of the safety margin in the joints.

Crack formation under peak loads :

When the structure was subjected to peak loads which are about 33% more than working load, the moment in the beam at critical section reached about 85 to 90% of its designed ultimate capacity and very wide cracks were observed. Even at the working load the moment in the beam reached about 65 to 70% of its capacity and clearly visible cracks have appeared. Therefore to avoid this excess cracking, it is not advisable to adopt a beam load factor less than about 1.4.

4.3 Theoretical study on the effect of joint load factors :

Joint load factor plays an important role in the load factor design. Theoretical analysis made on the effect of joint load factor showed that the overall load factor continues to increase with the increase in joint load factor upto a certain level beyond which it becomes constant. Reaching this constant position depends upon the ~~the~~ order of indeterminacy of the structure. As the order of indeterminacy increases, this constant position of overall load factor is reached at lower levels. The strength of structure increases due to increase in joint

load factor But at the same time cost also increases. It has been observed from the graphs that the difference in the numerical values of the percent increases in strength and cost, increases upto a certain value of joint load factor and then decreases. Because the strength and cost are two dissimilar quantities, this behaviour does not however represent any optimal condition. Increase in cost per unit increase in strength was found to be an increasing quantity without any peak or reduction in its value within the reasonable values of joint load factor.

Though there is an increase in the safety margin of the structure with the increase in joint load factor, the cost also will be increasing correspondingly. Sometimes it may not be desirable to increase the safety margin beyond certain level at the cost of economy. As such, depending upon the type of structure and the users' requirements, an optimal selection is to be made as to how much increase in cost can be permitted to obtain higher level of safety. Accordingly the load factors are to be chosen. In general the load factors of 1.4 to 1.5 for beams, 1.9 to 2.1 for joints and 1.7 for columns recommended earlier based on the experimental investigation

appear to be satisfactory with reference to the above criteria.

4.4 Future work :

More extensive experimental investigations on the safety and serviceability of the structure with many other parameters such as increased number of load cycles, different loading pattern on the structure, different load levels and different combination of load factors have to be conducted before arriving at a better choice of load factors for different elements in a load factor design. The effect of cyclic loading on the bond and shear needs to be investigated. Suitable loading pattern need be selected so that the failure mechanism includes formation of plastic hinges in the columns so as to incorporate the effect of column load factors. Simulation of actual loads, and the right choice of the normal working load and permissible excessive loads or reduced load factors at combined load conditions should also be investigated. This selection has a great influence on the economy of a structure.

APPENDIX - A

DESIGN OF SECTIONS FOR SPECIMEN 1

Ultimate Moment Capacity of the Beam of the Centre:

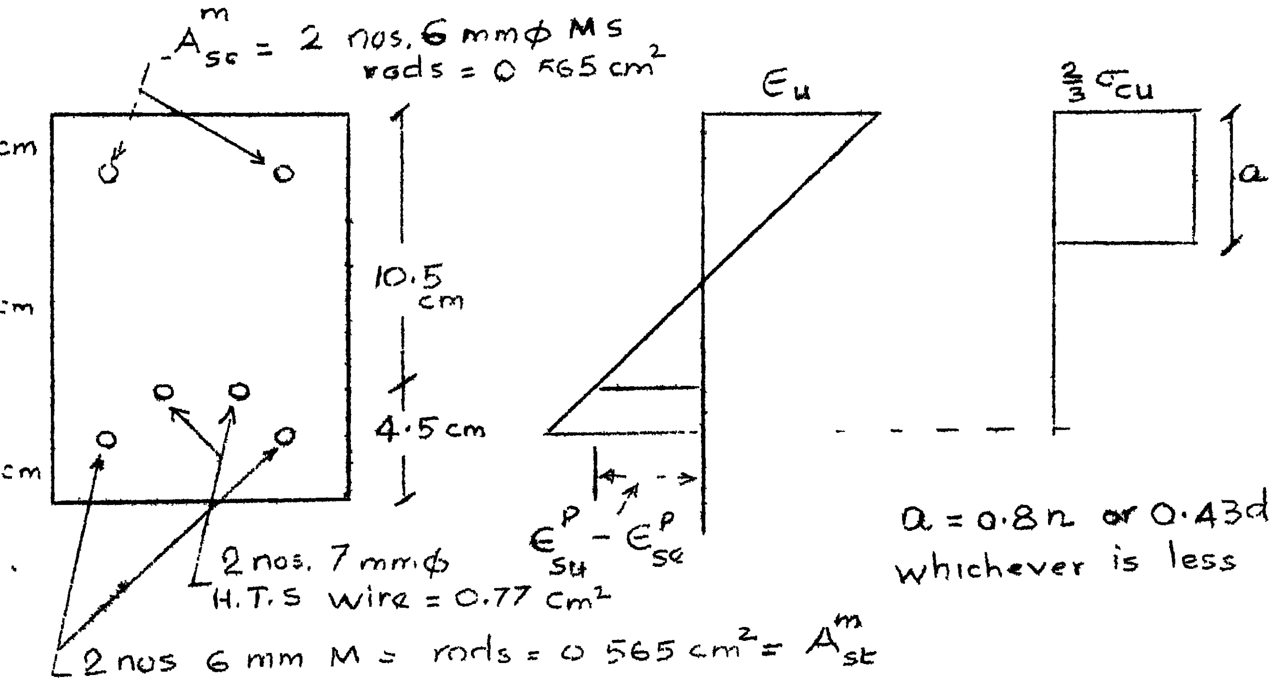


FIG. A-1 DETAILS OF BEAM SECTION

The compatibility equation is

$$\epsilon^{p}_{su} - \epsilon^{p}_{se} = \epsilon_u\left(\frac{d-n}{n}\right) \qquad (1)$$

where ϵ^{p}_{su} = Tensile strain in tendon at ultimate moment

ϵ^{p}_{se} = Effective strain in tendon after losses

ϵ_u = Ultimate strain in concrete

$$\epsilon^{P}_{se} = \frac{\sigma^{P}_{se}}{E_s} = \frac{0.6 \times \sigma_s}{E_s} = \frac{0.6 \times 16500}{2.1 \times 10^{6}}$$

$$= 4820 \times 10^{-6}$$

(σ_s' = ultimate stress of the High tension steel)

Using M - 450 concrete ϵ_u for doubly reinforced section is assumed as 5000×10^{-6}

$$\epsilon^{P}_{su} = \epsilon^{P}_{se} + \epsilon_u . \frac{(d-n)}{n}$$

$$= 4820 \times 10^{-6} + 5000 \times 10^{-6} \left(\frac{d-n}{n} \right)$$

$$= 4820 \times 10^{-6} \left[1 + 1.037 \left(\frac{d-n}{n} \right) \right]$$

value of n shall be found by trial and error.

Assume n = 4.7 cm

$$\epsilon^{F}_{su} = 4820 \times 10^{-6} \left\{ 1 + 1.037 \left(\frac{10.5 - 4.7}{4.7} \right) \right\}$$

$$= 10980 \times 10^{-6}$$

From the graph stress in steel when the strain is 10980×10^{-6} is

$$\sigma_{su} = 14650 \text{ kg/cm}^2$$

Strain in tension mild steel at ultimate load

$$= \epsilon_u \left(\frac{12 - n}{n}\right)$$

$$= 5000 \times 10^{-6} \left(\frac{12-4.7}{4.7}\right)$$

$$= 7750 \times 10^{-6}$$

Corresponding stress $= 7750 \times 10^{-6} \times 2.1 \times 10^{6}$

$$= 16300 \text{ kg/cm}^2 > \sigma_{sy}$$

$\therefore$ The mild steel in tension yields at the ultimate load.

Strain in compn. mild steel

$$= \epsilon_u \left(\frac{n-3}{n}\right)$$

$$= 5000 \times 10^{-6} \left(\frac{4.7-3}{4\ 7}\right)$$

$$= 5000 \times 10^{-6} \times \frac{1.7}{4.7}$$

$$= 1740 \times 10^{-6}$$

Corresponding stress in Compn. steel

$$= 1740 \times 10^{-6} \times 2.1\text{x}10^{-6}$$

$$= 3650 > \sigma_{cy}$$

$\therefore$ Compn. ~~mild~~ steel also yields at ultimate load.

Total tension in the section

$$T_u = A_{st}^{u} \times \sigma_{su} + A_{st}^{m} \times \sigma_{sy} \qquad (2)$$

$$= 0.77 \times 14650 + 0.565 \times 3200$$

$$= 11280 + 1810 = 13090 \text{ kg.}$$

Total compn. in the section

$$C_u = b \times a \times 2/3 \; \sigma_{cu} + A_{sc}^{m} \times \sigma_{cy} \qquad (3)$$

$$= 10 \times 0.8 \times 4.7 \times 2/3 \times 450 + 0.565 \times 3200$$

$$= 11300 + 1810 = 13110$$

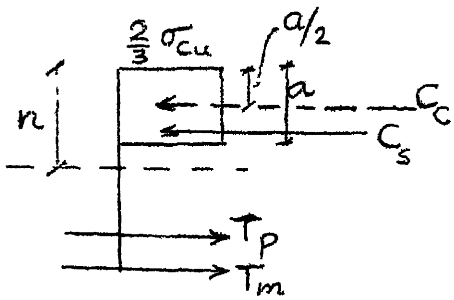


FIG. A-2.

$$T_u \simeq C_u$$

$\therefore$ Adopt $n = 4.7$ cm.

Position of the total compn. from top

$$= \frac{11300 \times 1.88 + 1810 \times 3}{13110} = \frac{21250 + 5430}{13110}$$

$$= \frac{26680}{13110} = 2.04 \text{ cm}$$

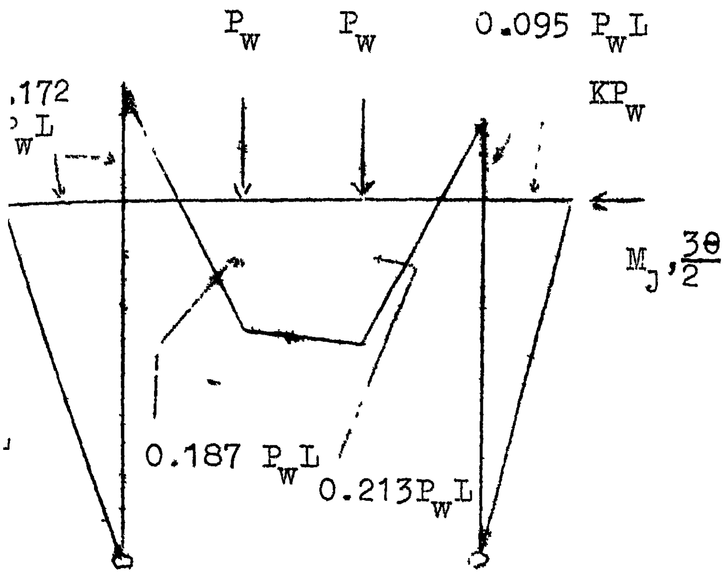
$$M_u = 11280 \times (10.5 - 2.04) + 1810 \times (12 - 2.04)$$

$$= 11280 \times 8.46 + 1810 \times 9.96$$

$$= 95400 + 18000 = 113400$$

Ultimate moment capacity of the beam section

$$= 113,400 \text{ kg.cm.}$$



K=1/13 for
specimen 1

FIG. A.3 B.M Diagram
for the specimen

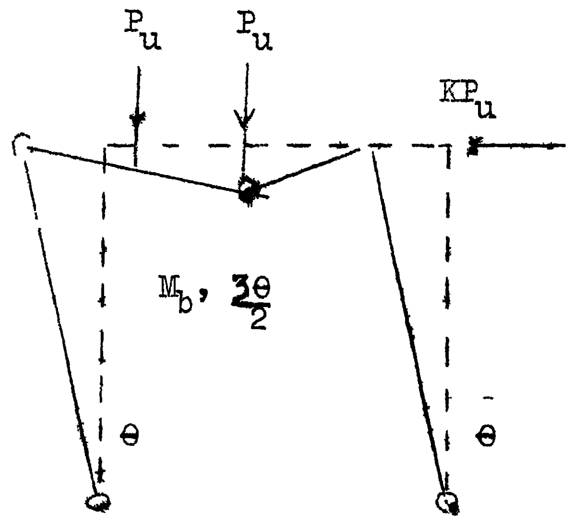


FIG.a.4 Collapse
Mechanism

CALCULATION OF ULTIMATE LOAD FROM COLLAPSE MECHANISM:

Load particulars of the frame and the bending moments at the salient points are given in Figure A-3. The combined mechanism failure which gives the least value of ultimate load on the specimen is shown in Fig. A-4.

By the principle of virtual work, equating the expressions for work done by the external forces and energy absorbed in the springs.

$$K\, P_u\, L\theta + P_u\, \theta\, \frac{L}{3} + P_u\, \frac{\theta}{2}\, \frac{L}{3} = (M_b + M_j)\, \frac{3\theta}{2}$$

or $$P_u = \frac{3(M_b + M_j)}{(1+2K)L} \quad (4)$$

OVERALL LOAD FACTOR:

Max. B.M. in the beam = $0.213\ P_w L$

Taking the load factor of the beam as 1.5

Ultimate moment in the beam $M_b = 1.5 \times 0.213\ P_w L$

$= 0.320\ P_w L$

Ultimate moment capacity of the section = 113.4 t.cm.

$0.320\ P_w\ L = 113.4$ Substituting $L = 150$ cm

Working load $P_w = 2.36$ t.

Maximum B.M at the joint = $0.172\ P_w L$

Taking the load factor of the joint as 2.2, the ultimate moment at the joint $M_j = 2.2 \times 0.172\ P_w L$.

$= 0.379\ P_w L$

$$\frac{M_j}{M_b} = \frac{0.379\ P_w L}{0.320\ P_w L} = 1.185 \quad (5)$$

From eqn. (4) and (5), $P_u = \dfrac{3(1.185\ M_b + M_b)}{(1 + \frac{2}{13})\ 150}$

Substituting the ultimate moment capacity of the beam of 113.4 t cm for M_b,

$$P_u = \frac{3 \times 2.185 \times 113.4}{1.154 \times 150} = 4.30 \text{ t.}$$

$$\text{Overall load factor } F_o = \frac{P_u}{P_w} = \frac{4.30}{2.36} = 1.82$$

Design of Column Section:

Max. B.M. in the column $= 0.172\ P_w L$

Taking load factor for the column as 1.7,

Ultimate moment in the column $= 1.7 \times 0.172\ P_w L$

$= 1.7 \times 0.172 \times 2.36 \times 150$

$= 103.5$ t.cm.

Ultimate axial load on the column

$= 1.7\ (\frac{14}{13}\ P_w)$

$= 1.7(\frac{14}{13} \times 2.36) = 4.32$ t.

$$\text{Eccentricity} = \frac{\text{Moment}}{\text{Axial load}} = \frac{103.5}{4.32} = 24 \text{ cm}$$

DETAILS OF THE COLUMN SECTION ARE GIVEN IN FIGURE A-5:

Assuming that both compression and tension steel yield at failure load, equilibrium equations are as follows:

Force equilibrium,

$$P' = \frac{2}{3}\sigma_{cu}\ b\ a + A^m_{sc}\ (\sigma_{cy} - \frac{2}{3}\ \sigma_{cu}) - A^m_{st}\ \sigma_{sy} \tag{6}$$

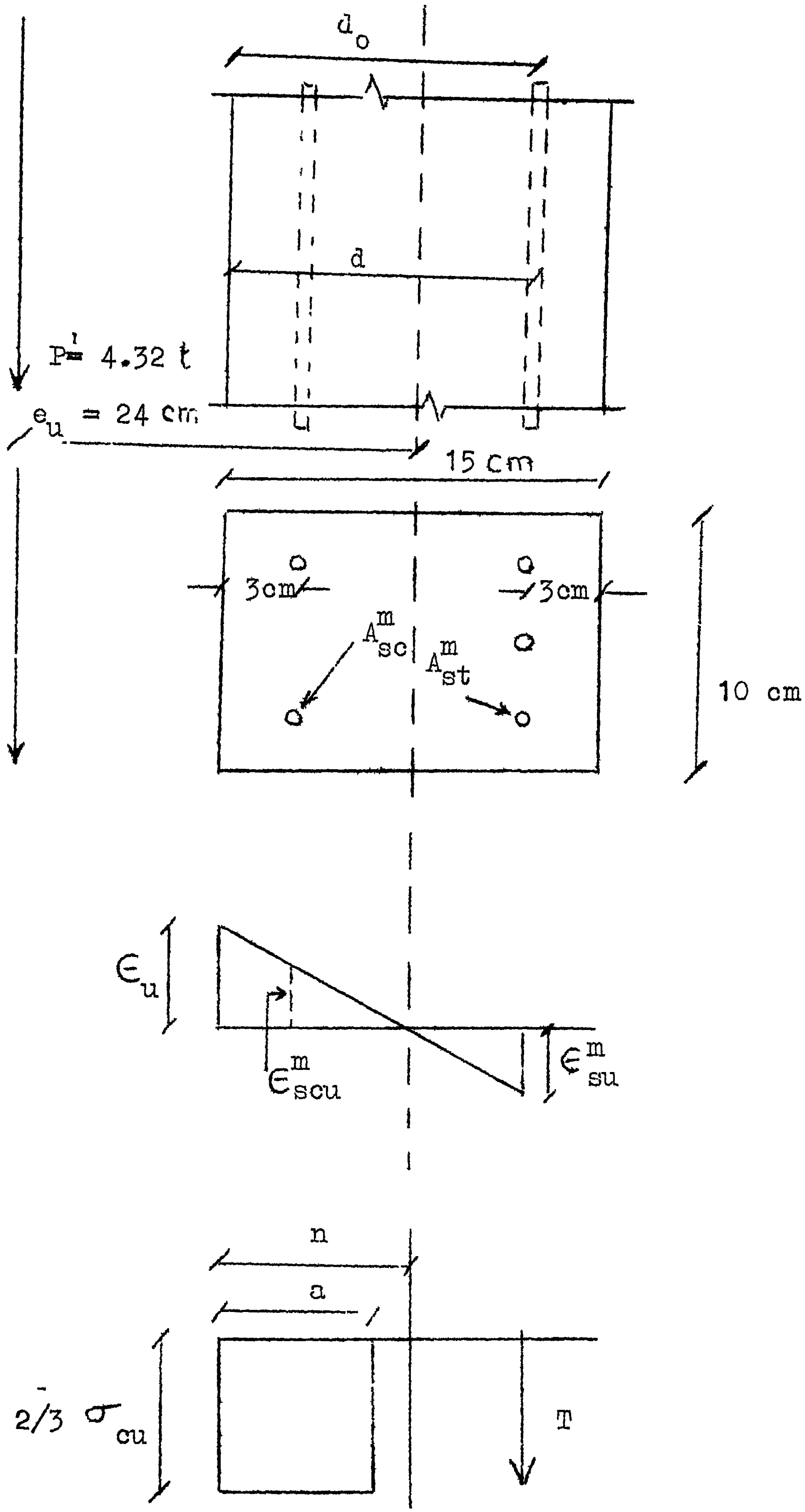


FIG. A-5 DETAILS OF COLUMN SECTION

Taking moments about the centre of tensile steel, the moment equilibrium equation is

$$P' \left(e_u + \frac{d_o}{2} - 3 \right) = \frac{2}{3} \sigma_{cu} \; a \; b \left(d - \frac{a}{2} \right) +$$

$$+ A_{sc}^{m} \left(\sigma_{cy} - \frac{2}{3} \sigma_{cu} \right) (d - 3) \qquad (7)$$

Compatibility equations are

$$n = \frac{\epsilon_u}{\epsilon_u + \epsilon_{su}^{m}} \; d \qquad (8)$$

$$\epsilon_{scu}^{m} = \left(1 - \frac{dc}{n} \right) \epsilon_u \qquad (9)$$

where P' = equivalent eccentric load

e_u = eccentricity at ultimate load

d_o = depth of section

b = width of section

a = depth of stress block

d = effective depth of section

σ_{cu} = cube strength of concrete

σ_{cy} = yield stress in steel in compression

σ_{sy} = yield stress in steel in tension

ϵ_{scu}^{m} = strain in compn. mild steel at the ultimate load.

From Eqn. (8)

$$n = \frac{5000 \times 10^{-6} \times 12}{5000 \times 10^{-6} + \frac{3200}{2.1 \times 10^{-6}}}$$

$= 9.2$ cm.

$a = 0.8\,n$ or $0.43\,d$ whichever is less

$0.8n = 0.8 \times 9.2 = 7.35$ cm

$0.43d = 0.43 \times 12.0 = 5.16$ cm

$a = 5.16$ cm

From Eqn. (9)

$$\epsilon^{m}_{scu} = \left(1 - \frac{3}{9.2}\right) 5000 \times 10^{-6}$$

$$= 3370 \times 10^{-6}$$

$$\sigma_{scu} = \epsilon^{m}_{scu} \times E$$

$$= 3370 \times 10^{-6} \times 2.1 \times 10^{6} = 7080 \text{ kg/cm}^2$$

which is greater than σ_{sy}, and hence the assumption that the compn. steel yields is correct.

From Equation (7)

$$4320\,(24 + 7.5 - 3) = \frac{2}{3} \times 450 \times 5.16 \times 10\left(12 - \frac{5.16}{2}\right)$$

$$+ A^{m}_{sc}\ \left(3200 - \frac{2}{3} \times 450\right)(12 - 3)$$

$$4320 \times 28.5 = 145,700 + 26100\, A_{sc}^{m}$$

$$123000 = 145,700 + 26100\, A_{sc}^{m}$$

From this A_{sc}^{m} is negative and is taken as zero.

Putting $A_{sc}^{m} = 0$ in eqns. (6) and (7)

$$P' = \frac{2}{3}\, \sigma_{cu}\, b\, a - A_{st}^{m}\, \sigma_{sy} \qquad (10)$$

$$P' \left(e_u + \frac{do}{2} - 3 \right) = \frac{2}{3}\, \sigma_{cu}\, b\, a \left(d - \frac{a}{2} \right) \qquad (11)$$

From Equation (11)

$$4320 \, (24 + 7.5 - 3) = \frac{2}{3} \times 450 \times 10 \times a \left(d - \frac{a}{2} \right)$$

$$123000 = 3000\, a \left(12 - \frac{a}{2} \right)$$

$$123 = 36\, a - 1.5\, a^2$$

$$a = 4.1 \text{ cm} < 0.43\, d$$

From Equation (10)

$$4320 = 2/3 \times 450 \times 10 \times 4.1 - A_{st}^{m} \times 3200$$

$$A_{st}^{m} = 2.50 \text{ cm}^2$$

Three bars of 12 mm giving 3.39 cm^2 area are provided on tension side. On compression side nominal reinforcement of two bars of 6 mm dia. are provided.

DESIGN OF JOINT SECTION ON COLUMN SIDE:

Ultimate moment at the joint with a load factor of 2.2

$$= 2.2 \times 0.172\ P_w L$$

$$= 2.2 \times 0.172 \times 2.36 \times 150$$

$$= 134.0 \text{ t.cm.}$$

Axial force $= 2.2 \left(\frac{14}{13} P_w\right)$

$$= 2.2 \times \frac{14}{13} \times 2.36 = 5.6 \text{ t}$$

Eccentricity $e_u = \frac{134.0}{5.6} = 24$ cm

$$a = 0.43 \times 12 = 5.16 \text{ cm}$$

Details of the section are some as in Figure A-5 except the value of P'.

From eqn. (7)

$$5600\ (24 + 7.5 - 3) = 2/3 \times 450 \times 5.16 \times 10\left(12 - \frac{5.16}{2}\right)$$

$$+ A_{sc}^{m}\ \left(3200 - \frac{2}{3} \times 450\right)\ (12-3)$$

$$159500 = 145{,}700 + 28{,}800\ A_{sc}^{m}$$

$$A_{sc}^{m} = 0.48 \text{ cm}^2$$

From eqn. (6)

$$5600 = \frac{2}{3} \times 450 \times 10 \times 5.16 + 0.48\ (3200-\frac{2}{3} \times 450) - A_{st}\ \sigma_{sy}$$

$$5600 = 15480 + 1390 - A^{m}_{st} \times 3200$$

$$A^{m}_{st} = 3.52\ cm^2$$

In addition to the 3 bars of 12 mm extended from the column into the joint portion an additional 6 mm bar is provided. Total area of reinforcement on tension side will be: $3.39 + 0.285 = 3.675\ cm^2$. On compression side two bars of 6 mm are extended from the column portion.

Design of Joint Section, Beam Side:

Ultimate moment at the joint $= 2.2 \times 0.172\ P_w L$

$= 134.0$ t cm.

Axial force $= 2.2 \times 0.172\ P_w$

$= 2.2 \times 0.172 \times 2.36 = 0.892$ t

Moment cpacity of the joint required = 134.0 t.cm.

Moment capacity provided by the beam section = 113.4 t.cm.

Balance to be provided for = 134.0 - 113.4 = 20.6 t.cm.

$$\text{Eccentricity} = \frac{20.6}{0.892} = 23.1\ \text{cm.}$$

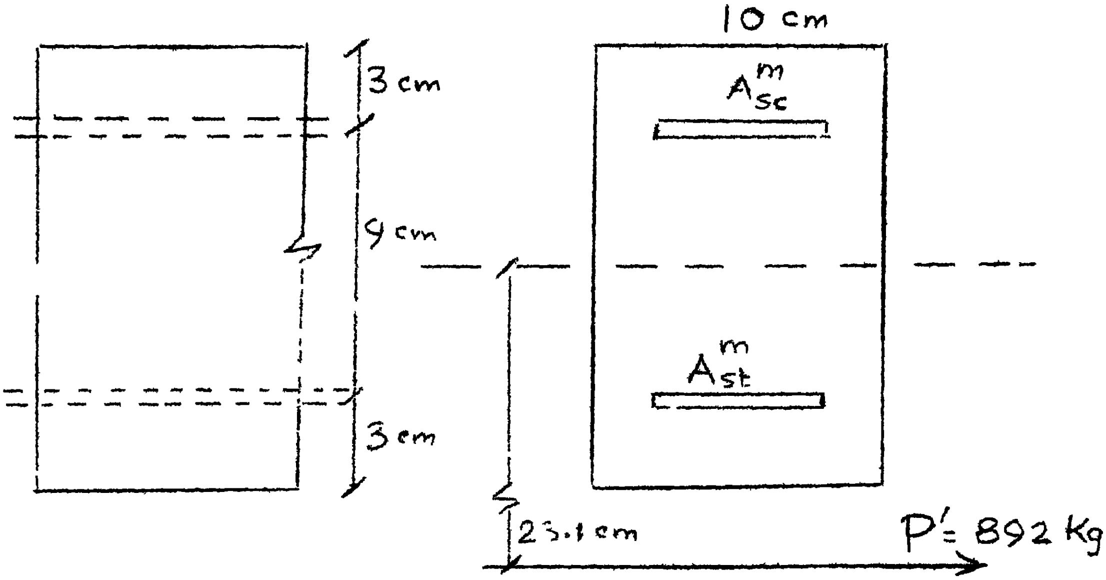


FIGURE A-6 EXTRA REINFORCEMENT AT THE JOINT ON BEAM-SIDE.

Assuming that both compn. and tension steel yield at failure load from force equilibrium condition,

$$892 = A^m_{sc} \times 3200 - A^m_{st} \times 3200 \qquad (12)$$

From moment equlibrium condition, taking moments about the centre of tension steel,

$$892\ (23.1 + 4.5) = A^m_{sc} \times 3200 \times 9$$

$$A^m_{sc} = 0.854 \text{ cm}^2$$

From eqn. (12)

$$(A_{sc}^{m} - A_{st}^{m})\ 3200 = 892$$

$$(0.854 - A_{st}^{m}) = \frac{892}{3200} = 0.278$$

$$A_{st}^{m} = 0.575\ cm^2$$

Alternately, if the axial force is neglected, the reinforcement required at top and bottom to take the additional moment of 20.6 t cm is

$$A_{st}^{m} \times 3200 \times 9 = 20.6 \times 1000$$

$$A_{st}^{m} = A_{sc}^{m} = \frac{20.6 \times 1000}{3200 \times 9} = 0.723\ cm^2$$

Therefore one bar of 12 mm dia is provided at top and bottom in addition to the reinforcement extended from the beam.

DESIGN OF SHEAR REINFORCEMENT IN THE BEAM:

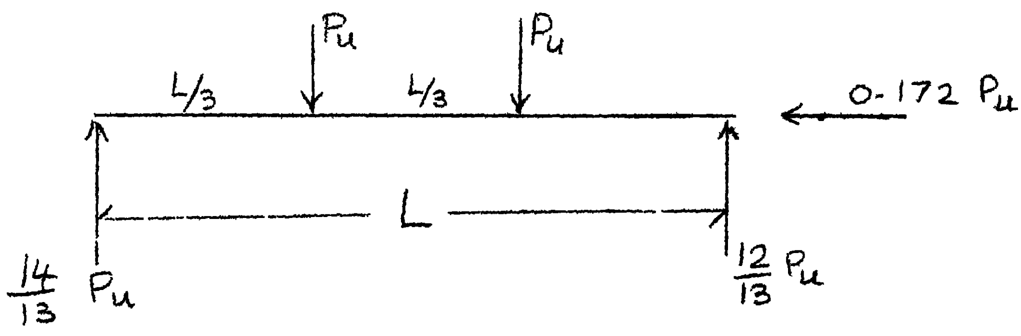


FIGURE A-7 FREEBODY DIAGRAM OF THE BEAM PORTION

Maximum shear in the beam = $\frac{14}{13} P_u$

$= 1.08 \times 4.30 = 4.63$ t

The section is also subjected to axial compression

$= 0.172 \, P_u = 0.74$ t.

Failure against shear is provided by adopting the safety factor for shear as 1.2 times the load factor of the total failure mechanism.

Desired shear capacity = $1.2 \times 4.63 = 5.57$ t.

Allowable shear stress as per ACI Code in concrete

$$q_c = 0.79 \sqrt{0.8 \, \sigma_{cu} \left(1 + 0.025 \frac{P}{A_o}\right)} \text{ kg/sq.cm.}$$

where P = axial load, plus if compn. and minus if tension

A_o = gross area of section = 150 sq.cm.

$$q_c = 0.79 \sqrt{0.8 \times 450 \left(1 + 0.025 \times \frac{740}{150}\right)}$$

$$= 15.9 \text{ kg/cm}^2$$

Allowable shear force on the section without shear reinforcement = $15.9 \times 150 = 2390$ kg.

Shear force to be provided by shear reinforcement

$\simeq 5570 - 2390 = 3180$ kg.

C.S. area of vertical stirrups required

$$A_w = \frac{Q'_u \times s}{0.85 \sigma_{sy} d}$$

where Q'_u = ultimate shear to be carried by the web reinforcement alone

= 3180 kg.

s = spacing of stirrups

σ_{sy} = yield strength of web reinforcement = 3200 kg/cm^2

Using 6 mm dia. two legged stirrups $A_w = 0.283 \times 2 = 0.566$

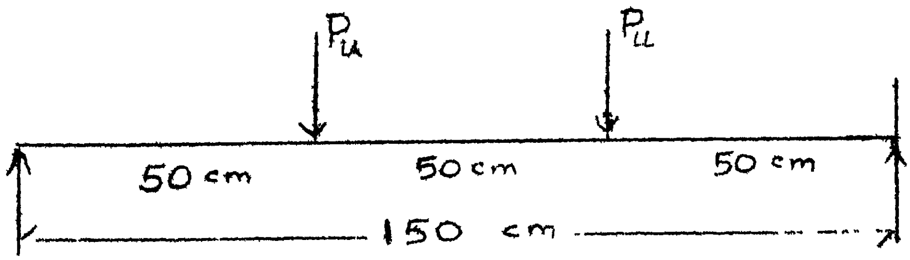
$$s = \frac{A_w \times 0.85 \sigma_{sy} d}{Q'_u}$$

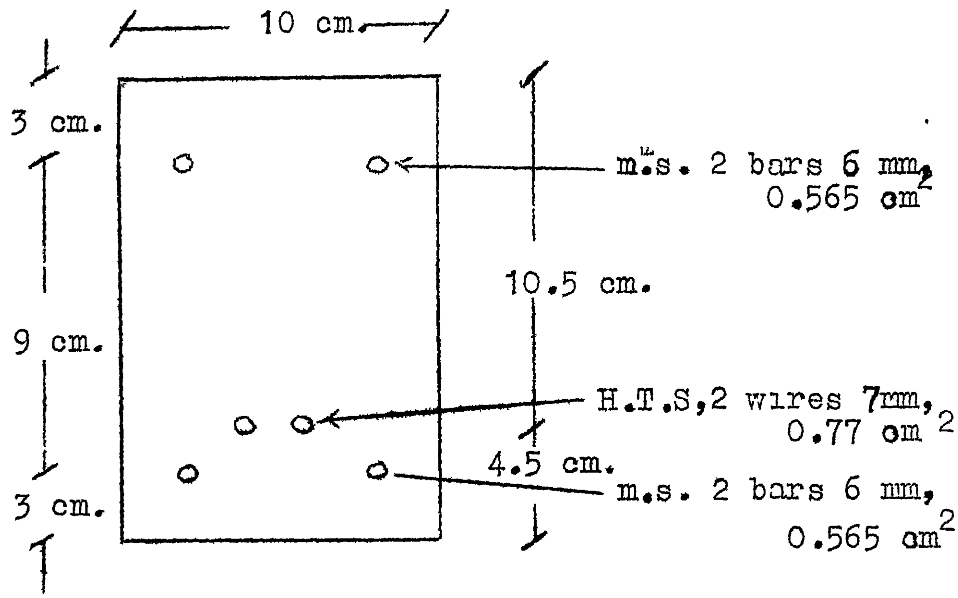
$$= \frac{0.566 \times 0.85 \times 3200 \times 12}{3180} = 7.8 \text{ cm say } 8 \text{ cm.}$$

APPENDIX - B

ULTIMATE LOADS ON SIMPLY SUPPORTED BEAMS

Beam - 1 : Simply supported unbonded prestressed concrete beam :





SECTION AT C&D

FIG. B-1 DETAILS OF SPAN, SECTION OF SIMPLY SUPPORTED BEAM

with the same notation as adopted in Appendix - A,

$$\epsilon_{su}^{p} = 4820 \times 10^{-6} \left(1 + 1.037 \frac{(d-n)}{n}\right)$$

value of n by trial and error method :

Try $n = 4.2$ cm

$$\epsilon_{su}^{p} = 4820 \times 10^{-6} \left(1 + 1.037 \frac{(10.5 - 4.2)}{4.2}\right)$$

$$= 12320 \times 10^{-6}$$

From graph (Figure 3.4) $\sigma_{su}^{p} = 14850$ kg/cm^2

Total tension $T_u = 0.77 \times 14850 + 0.565 \times 3200$

$$= 11420 + 1810 = 13230$$

Total compn. $C_u = 10 \times 0.8 \times 4.2 \times 2/3 \times 500 + 0.565 \times 3200$

$$= 11280 + 1810 = 13090$$

$$C_u \simeq T_u$$

$\therefore$ adopt $n = 4.2$ cm

Distance of C.G. of Total compression from top

$$= \frac{11280 \times 1.68 + 1810 \times 3}{11280 + 1810}$$

$$= 1.9 \text{ cm.}$$

Taking moments about the line of action of total compression

$$M_u = 11420 \times 8.6 + 1810 \times 10.1$$

$$= 98200 + 18300$$

$$= 116{,}500 \text{ kg. cm.} = 116.5 \text{ t.cm.}$$

Ultimate moment capacity of the beam section

$$= 116.5 \text{ t.cm.} \qquad (1)$$

Max bending moment in the beam at C or D

$$= P_u \times \frac{L}{3} = \frac{150}{3} P_u$$

$$= 50 P_u \text{ t.cm.} \qquad (2)$$

Equating the B.M. and moment capacity from (1) and (2)

$$50 P_u = 116.5$$

$$P_u = \frac{116.5}{50} = 2.33 \text{ t}$$

Therefore theoretical ultimate load on beam 1 = 2.33 t

Ultimate load obtained from the static load test on beam 1 = 2.24 t.

The theoretical ultimate load is found to tally with the experimental value, when no reduction factor for unbondedness is used. Therefore for all the calculation of ultimate moment capacities of the unbonded prestressed sections for the beam portion in all the frame specimens no reduction for unbondedness was used.

Simply supported beam without prestressing, with H.T.S. reinforcement, bonded, (Beam-2)

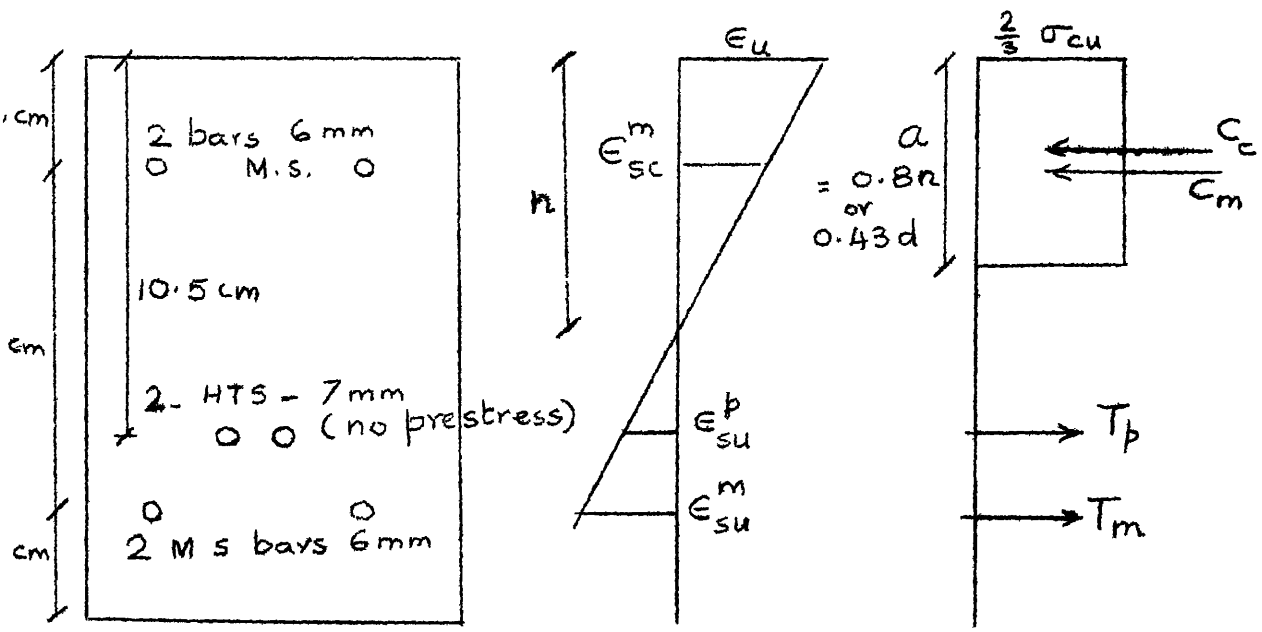


FIG. B-2 : DETAILS OF SECTION OF BEAM-2

Cube strength : 450 kg/cm^2

Value of n by trial & error :

Try n = 4.25 cm

$$\epsilon_u = 5000 \times 10^{-6}$$

$$\epsilon_{su}^{p} = 5000 \times 10^{-6} \ \frac{(10.5-4.25)}{4.25}$$

$$= 7350 \times 10^{-6}$$

From graph (Figure 3.4), $\sigma_{su}^{p} = 13200$ kg/cm^2

$$\epsilon_{su}^{m} = 5000 \times 10^{-6} \ \left(\frac{12-4.25}{4.25}\right)$$

$$= 9130 \times 10^{-6}$$

From graph (Figure 3.5) $\sigma_{su}^{m} = 3200$ kg./cm^2

$$\epsilon_{sc}^{m} = 5000 \times 10^{-6} \left(\frac{4.25-3}{4.25}\right)$$

$$= 4700 \times 10^{-6}$$

From graph (Fig. 3.5), $\sigma_{sc}^{m} = 3200$ kg./cm^2

Total tension T_u = 0.77x13200 + 0.565x3200

= 10,170 + 1810 = 11,980 kg.

Total compn. C_u = 10x0.8x4.25x2/3x450+0.565x3200

= 10200+1810 = 12,010 kg.

$$T_u \simeq C_u$$

$\therefore$ adopt n : 4.25 cm

a = 3.4 cm

c.g. of total compn. from top :

$$= \frac{10200 \times 1.7 + 1810 \times 3}{12010}$$

$\simeq$ 1.9 cm.

Taking moments about the position of total compression

M_u = 10170 (10.5-1.9) + 1810 (12-1.9)

= 87,500 + 18,300

= 105,800 kg. cm. (3)

Max B.M. in the beam at C or D = $P_u \times \frac{150}{3}$

= 50 P_u tcm (4)

From (3) and (4) $P_u = \frac{105.8}{50} = 2.12$ t

$\therefore$ Theoretical value of ultimate load on the beam

$$P_u = 2.12 \text{ t}$$

Ultimate load obtained by static load test on

beam - 2 (from Fig. 3.17) : 2.4 T.

Therefore in this case also the theoretical and experimental ultimate load values are closely agreeing.

APPENDIX - C

RE-EVALUATION OF ULTIMATE STRENGTH OF SPECIMENS BASED ON ACTUAL CUBE STRENGTH OF SPECIMEN-1

Tested cube strength of elements = 540 kg/cm^2

Tested cube strength of joints = 517 $kg./cm^2$

Ultimate moment of beam section

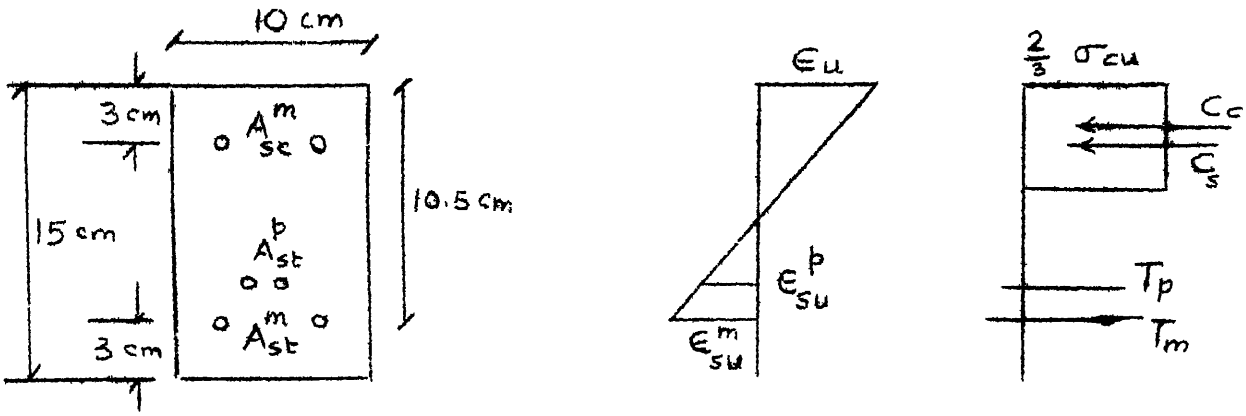


FIG. C.1 DETAILS OF BEAM SECTION

with the same notation as that in Appendix - A.

$$\epsilon^p_{su} = 4820 \times 10^{-6} \left(1 + 1.037 \ \frac{(d-n)}{n} \right)$$

Value of n by trial and error method :

Try n = 4.1 cm

$$\epsilon_{su}^{p} = 4820 \times 10^{-6} \left[1 + 1.037 \; \frac{(10.5-4.1)}{4.1} \right]$$

$$= 12600 \times 10^{-6}$$

from graph (Fig. 3.4), $\sigma_{su}^{p} = 14900 \text{ kg/cm}^2$

Total tension $T_u = 0.77 \times 14900 + 0.565 \times 3200$

$= 11470 + 1810 = 13,280$ kg.

Total compression $C_u = 10 \times 0.8 \times 4.1 \times 2/3 \times 540 + 0.565 \times 3200$

$= 11800 + 1810 = 13,610$ kg.

$$T_u \approx C_u$$

Therefore adopt n = 4.1 cm

Distance of centre of gravity of the total compression from the top edge

$$= \frac{11800 \times 1.64 + 1810 \times 3}{13610} = 1.82 \text{ cm.}$$

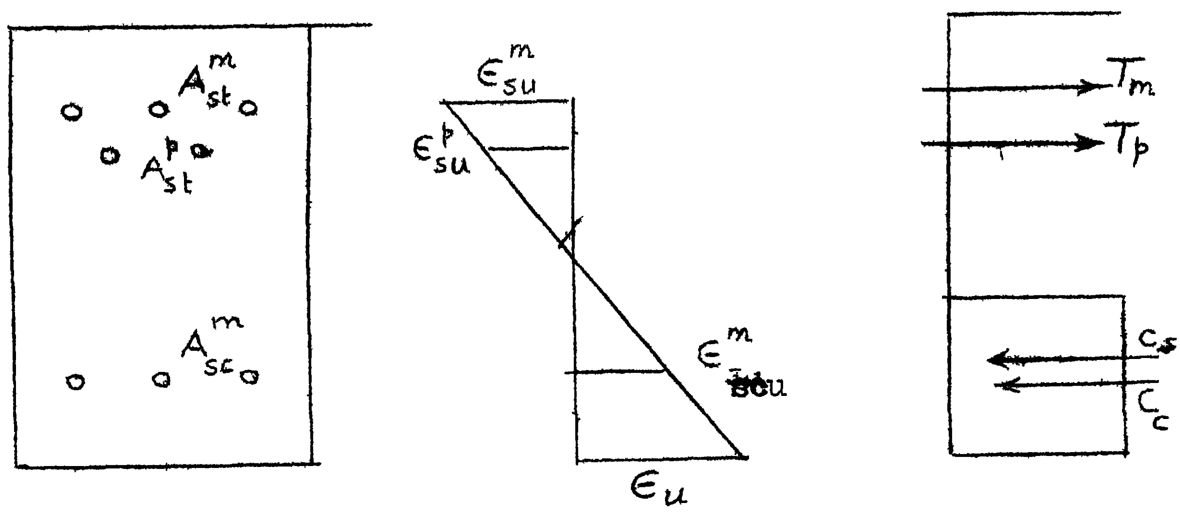
Taking moments about the position of total compression

$$M_u = 11470\ (10.5-1.82) + 1810\ (12.0-1.82)$$

$$= 99{,}500 + 18440 = 117{,}940 \text{ kg. cm.}$$

<u>Ultimate moment capacity of joint on beam side</u>

Cube strength of concrete of the joints = 517 kg./cm^2



A^m_{st} = 3 bars M.S., 2 of 6 mm + 1 of 12 mm

A^p_{st} = 2 wires H.T.S, 7 mm dia.

A^m_{sc} = 2 bars 6 mm + 1 bar 12 mm of M.S.

FIG. C-2 : DETAILS OF JOINT SECTION ON BEAM SIDE

Try n = 4.15 cm :-

$$a = 0.8 \times 4.15 = 3.320 \text{ cm}$$

$$\epsilon_{su}^{p} = 4830 \times 10^{-6} \left[1 + 1.037 \; \frac{(10.5-4.15)}{4.15} \right]$$

$$= 12450 \times 10^{-6}$$

From graph (Fig. 3.4),

$$\sigma_{su}^{p} = 14850 \text{ kg/cm}^2$$

Total tension T_u = 0.77 x 14850 + 1.695 x 3200

= 11420 + 5420 = 16840 kg.

Total compression C_u = 10x0.84x4.15x2/3x517+1.695x3200

= 11440 + 5420 = 16860 kg.

$$T_u \approx C_u$$

$\therefore$ adopt n = 4.15 cm

C.G. of compression force from bottom

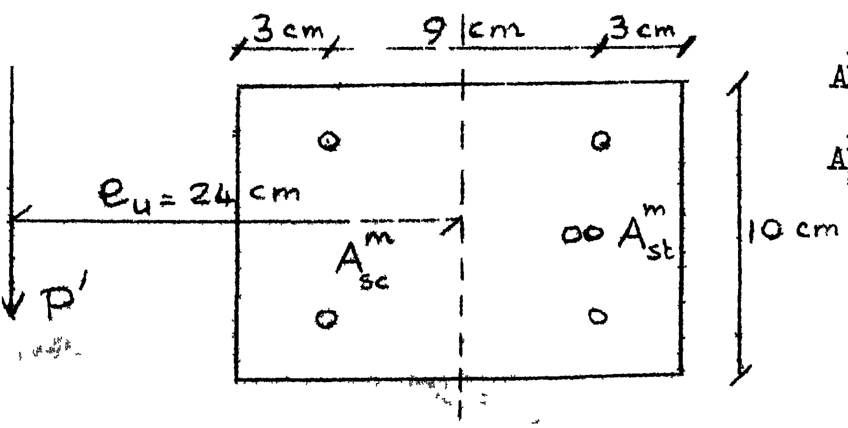
$$= \frac{11440 \times 1.66 + 5420 \times 3}{16860} = 2.09 \text{ cm}$$

$$\therefore M_u = 11420\ (10.5-2.09) + 5420\ (12-2.09)$$

$$= 96200 + 53{,}700 = 149{,}900 \text{ kg. cm.}$$

Ultimate moment capacity of the joint on column side

Cube strength = 517 kg./cm^2.



A^m_{sc} = 2 bars M.S. of 6 mm

A^m_{st} = M.S. 3 bars of 12 m
+ 1 of 6 m

FIG. C-3 : DETAILS OF JOINT SECTION ON COLUMN SIDE

Using the same notation as in Appendix A,

Try n = 5.8 cm :-

$$a = 0.8 \times 5.8 = 4.64 \text{ cm}$$

$$\epsilon^m_{scu} = 5000 \times 10^{-6} \times \left(\frac{5.8-3}{5.8}\right)$$

$$= 2420 \times 10^{-6}$$

From graph (Figure 3.5)

$$\sigma^{m}_{scu} = 3150 \text{ kg/cm}^2$$

From eqn (6) of Appendix - A

$$P = 2/3 \times 517 \times 4.64 \times 10 + 0.565 \times 3150$$

$$- 3.673 \times 3200$$

$$= 16000 - 1780 - 11750 = 6030 \text{ kg.}$$

From eqn. (7) of Appendix A

$$(24+7.5-3)\ P' = 16000\ (12 - \frac{4.64}{2}) + 1780 \times 9$$

$$= 171,000$$

$\therefore$ $P' = 6000$ kg.

$\therefore$ The value of P' obtained from the two formulae is approximately tallying.

Therefore adopt n : 5.8 cm.

$M_u = 6000 \times 24 = 144,000$ kg. cm.

$\therefore$ Moment capacity of joint section on beam side = 144.0 t.cm.

Load factors

Ultimate moment capacity of the beam = 117.9 t.cm.

Ultimate moment capacity of the joint on beam side = 149.9 t.cm.

Ultimate moment capacity of the joint on column side = 144 t.cm.

$\therefore$ M_b = 117.9 t.c m., M_j = 144.0 t.cm.

Beam load factor $F_b = 1.5$

$1.5 \times 0.213\ P_w L\ :\ 117.9$

$$\therefore P_w = \frac{117.9}{1.5 \times 0.213 \times 150} = 2.46 \text{ t.}$$

$F_j \times 0.172\ P_w L = 144.0$

$$\therefore F_j = \frac{144.0}{0.172 \times 2.46 \times 150} = 2.27$$

Theoretical ultimate load carried by the frame

$$P_u^T = \frac{3\ (M_b + M_j)}{(1+2K)\ L}$$

$$= \frac{3(117.9+144.0)}{(1+2x\frac{1}{13})\ 150} = 4.53 \text{ t}$$

$$\therefore \text{ overall load factor } = \frac{P_u^T}{P_w} = \frac{4.53}{2.46} = 1.84$$

<u>Load factors reevaluated based on the experimental values of ultimate load on the specimen - 1 obtained by the experiment</u> :

$$P_u^E = 5.45 \text{ t.}$$

working load applied on the specimen during the experiment

$$P_w = 2.46 \text{ t}$$

$$\frac{P_u^E}{P_u^T} = \frac{5.45}{4.53} = 1.20$$

Since the working load is same and the ultimate is changed the revised load factors are obtained multiplying the earlier values by 1.20

Revised value of overall load factor $F_o = 1.84x1.2=2.21$

$$F_b = 1.2 \times 1.5 = 1.8$$

$$F_j = 1.2 \times 2.27 = 2.72$$

Load factors obtained on the basis of theoretical ultimate load are given in Table 3.4 and those based on the experimental ultimate load are given in Table 3.5.

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